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FLOOD CONTROL

CONNECTICUT RIVER VALLEY

REPORT OF SURVEY
AND
COMPREHENSIVE PLAN

APPENDIX, VOLUME I

SECTION 1 - HYDROLOGY AND METEOROLOGY
SECTION 2 - FLOOD LOSSES - BENEFITS
SECTION 3 - CONSERVATION - POWER AND RECREATION

UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

FLOOD CONTROL

CONNECTICUT RIVER VALLEY

REPORT OF SURVEY

AND

COMPREHENSIVE PLAN

UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

APPENDIX, VOLUME 1

SECTION 1 - HYDROLOGY AND METEOROLOGY
SECTION 2 - FLOOD LOSSES, - BENEFITS
SECTION 3 - CONSERVATION - POWER AND RECREATION

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FLOOD CONTROL

CONNECTICUT RIVER VALLEY

REPORT OF SURVEY

AND

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HYDROLOGY AND METEOROLOGY

SECTION 1 OF THE APPENDIX

(VOLUME 1)

HYDROLOGY AND METEOROLOGY

1. Scope.- There are presented in this section of the Appendix hydrological data and its sources for the Connecticut River Basin during recent major floods; probable flood frequencies derived from all data of record; methods used in the flood control analysis; the flood reducing effect of the reservoirs investigated; the considerations governing and the determination of the sizes of spillways and outlets, and the type of outlet control; and the probable frequency with which water will rise to various elevations within each reservoir. Reference is made to the Main Report for an extended treatment of the hydrology and flood history of the Connecticut River Basin. Although many additional reservoirs were investigated, the data referring to specific reservoirs herein are limited to the thirty forming the Comprehensive Plan of Reservoirs and their alternates.

2. Sources of hydrological data.- Rainfall data at points in and near the Connecticut River Basin are collected by the United States Weather Bureau, the Meteorological Service of Canada, the Massachusetts Department of Public Health, and the Connecticut Ground Water Survey. Data were obtained from these sources for all rainfall stations listed in Table 1. Records of depth and density of snow cover are obtained by the New England Power Company at the following stations in Vermont: Comerford, Ellis Brook, Harriman, Marshfield, Peacham, Searsburg Mountain, Searsburg Station, Somerset, and West Burke; and at First Connecticut Lake and York Pond in New Hampshire. Records of depth of snow cover were received from the Connecticut River Hydrological Survey for the following stations in Vermont: Bellows Falls, Brookfield, Burke Mountain, Canaan, Danville, Lunenburg, Rochester, Tunbridge, Lyson, Wells River, West Topsham, White River Junction, and Wilmington. Snow-

cover records for several additional points in the watershed were available from the United States Weather Bureau. The locations of all rainfall and snow-cover stations are shown on Plate No. 1. Records of stream run-off at eight gaging stations on the Connecticut River and thirty-seven gaging stations on tributaries are available in publications and files of the Water Resources Branch of the United States Geological Survey. A list of all existing and discontinued stream-gaging stations in the Connecticut River Basin and the period of record of each station are given in Table 2. The locations of the existing stations are shown on Plate No. 1. Stage hydrographs at several additional points on the Connecticut River were obtained from the New England Power Company and several other sources. Rating curves at stream-gaging stations on the Connecticut River are shown on Plate No. 2 and rating curves at principal tributary gaging stations are shown on Plate No. 3. Tentative adjustments have been applied to the extensions of the well-defined portions of several of the rating curves by utilizing data on flood volumes, high-water elevations, and estimates of peak discharges by the distribution-graph method and by channel-discharge computations. In some cases, such as Hartford, where the failure of dikes affected the relation of stage to discharge, the rating curves were computed as they would have been without these conditions in order to make stage reductions by reservoirs, taken therefrom, approximately as they would be during future floods.

3. Hydrological data for November 1927 flood.- Rainfall graphs from recording rainfall records at sixteen stations in and near the Connecticut River Basin are shown on Plate No. 4 for the period, November 2 - 5, inclusive, 1927. The rainfall map for the same period was prepared from all the available rainfall records and is shown on Plate No. 5. Discharge hydrographs for the period, November 3 - 12, 1927, at seven gaging stations on the Connecticut River and at 12 points on

tributaries are shown on Plate No. 6. The volumes of run-off and peak discharges from these hydrographs are shown on Table 3. Watershed maps and the high-water profiles of the November 1927 flood on the Connecticut River and principal tributaries are shown on Plates No. 164-193, inclusive.

4. Hydrological data for March 1936 flood.- Rain graphs from recording rainfall records at fourteen stations in the northeastern United States for the period, March 16-22, inclusive, 1936, are shown on Plate No. 7. The rainfall map for the same period is shown on Plate No. 8. Discharge hydrographs for the period, March 16-26, 1936 at seven gaging stations on the Connecticut River and at 12 points on tributaries are shown on Plate No. 9. The volumes of run-off and peak discharges from these hydrographs are shown on Table 3. The high-water profiles of the March 1936 flood on the Connecticut River and principal tributaries are shown on the profile plates.

5. Probable frequency of recurrence of peak discharge.- The frequency of peak discharges was determined at each gaging station in the Connecticut River watershed with a period of record longer than 12 years by the basic-stage method. The frequency equation used is $C = \frac{n}{m-0.5}$ in which C is the probable frequency of recurrence in years of a given value of discharge, m is the number of times during the period of record that the given discharge has been equalled or exceeded, and n is the number of years of record. Twenty-four hour average discharges were converted into instantaneous peak discharges and plotted against their frequency of recurrence and a smooth curve drawn through them. Curves of frequency thus determined are shown on Plates Nos. 10 and 11. Discharges at various parameters of frequency were read from each frequency curve, divided by the drainage area at the station and plotted to form a relation between frequency of instantaneous peak discharge and drainage area. It was found that the parameters of frequency conformed

closely to a smooth alignment for the Connecticut River Basin with the exception of that part south of the Ammonoosuc River and east of the Connecticut River, which conformed to a smooth alignment of its own. These relations are shown on Plate No. 11.

6. Probable frequency of recurrence of flood volume.- The flood volume versus frequency relations were determined at all gaging stations for which data were available by following the procedure outlined in the paragraph above, and the general relations of flood volume in inches of depth on each drainage area for various parameters of frequency were determined with drainage area as the other variable. The volume of a given flood was taken as the total discharge from the beginning of rise to the end of the flood period. It was found by a comparison of peak discharge and volume-frequency relations that for corresponding frequencies at any given station the ratio of peak discharge to flood volume is approximately a constant. The constancy of this ratio made possible, as explained later, the determination of peak discharge versus frequency relations for ungaged drainage areas. The relations of flood volume to frequency established from actual records are shown on Plates Nos. 10 and 11.

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(Report continued on following page)

DETERMINATION OF THE RUN-OFF HYDROGRAPH FROM RAINFALL

7. General description of method.- It was necessary in studies made for this report to reconstitute past floods from rainfall data and to construct hypothetical floods from assumed rainfall conditions. This was accomplished by use of the distribution-graph method which has proven to be the most accurate method yet developed for converting rainfall into run-off hydrographs. Distribution graphs were determined for 22 tributary watersheds of the Connecticut Valley for which stream-discharge data were available and were verified by reconstituting flood hydrographs of record. Properties of these distribution graphs were related to topographic features of their respective watersheds and general relations were established therefrom, thus providing a means by which distribution graphs could be obtained for drainage areas without discharge records. Distribution graphs for various increments of rainfall duration were derived for use in constructing spillway-design floods.

8. Reference to articles on the distribution graph and the unit hydrograph.- A complete discussion of the distribution graph and of the unit hydrograph is not given herein because detailed articles describing each have been presented in various technical publications. Reference is made to the following articles:

"Stream Flow from Rainfall by the Unit Graph Method," Engineering News-Record, Vol. 108, pp. 501-505, 1932, by L. K. Sherman.

"Approach to Determinate Stream Flow," Am. Soc. Civil Eng. Proc., Vol. 60, pp. 3-18, 1934, by M. M. Bernard.

"Surface Run-Off Phenomena," pt. 1, Analysis of the Hydrograph: Horton Hydrol. Lab., Pub. 101, Feb. 1, 1935.

"Studies of Relations of Rainfall and Run-Off in the United States," Geological Survey Water Supply Paper 772.

9. Definition of the unit hydrograph.- The "Unit Hydrograph" for a given watershed is the discharge hydrograph representing one-inch depth of surface run-off that would result from a storm of unit duration and of uniform intensity and distribution over the watershed. The theory of the unit hydrograph is based on the assumptions that the run-off from any storm of unit duration will follow the same regimen of flow as that of the unit hydrograph and, as a result, that the discharge ordinates will vary directly as the volumes of surface run-off. Accordingly, the total hydrograph from a storm of any duration may be pictured as the summation of a series of component hydrographs, one for each unit period of the storm. In developing a run-off hydrograph from rainfall by employing the unit-hydrograph method, the rainfall is broken down into unit periods; the volume of run-off in inches of depth for each period is estimated; a discharge hydrograph is constructed for each unit period of rainfall by multiplying the discharge ordinates of the unit hydrograph by the estimated volume of run-off for each period; and a composite hydrograph of surface run-off is obtained by adding these separate hydrographs. When ground-water inflow and run-off from prior storms are appreciable, they are estimated independently and added to the above hydrograph.

10. Definition of the distribution graph.- If a discharge period such as 12 hours is selected and the period-by-period ordinates of the unit hydrograph are expressed as percentages of their sum, these percentages, plotted versus time constitute the "Distribution Graph" for the watershed. The distribution graph was developed for each tributary watershed in the Connecticut Basin for which discharge data were available because its use permits comparison of graphs from various drainage areas regardless of size. The unit hydrograph was used, however, in establishing the relations with topographic features because it facilitated computations involving laws of hydraulic similitude, which were

required for one step in developing these relations.

11. Factors involved.- Once the rain has fallen on the ground, it first fills the small pockets in the terrain, and then flows overland towards the minor stream channels. During this portion of its journey, which Robert E. Horton has shown to take only a few hours, part of it is lost from direct surface run-off by evaporation, absorption, and infiltration. The amount of this loss is not treated herein because in reconstituting past floods for drainage areas without discharge records the percentages of run-off were estimated from a study of recorded volumes of run-off at nearby gaging stations, and in constructing spillway-design floods a maximum percentage of run-off was used. The variation in surface cover throughout the year affects the volumes of run-off from various storms but not the characteristics of the resultant hydrographs at points on the main stream channels. A hydrograph for a given drainage area is composed of the direct surface run-off into the many minor channels of a drainage system modified by open channel flow plus ground-water inflow. Open channel flow is affected chiefly by natural valley storage and the synchronization of hydrographs from various sub-areas. These two factors vary with topographic features such as size and shape of drainage area, ruggedness of terrain, slopes of the main stream beds, and the discharge characteristics of the stream channels. Thus, the unit hydrograph is a function of the topographic features of a watershed which remain relatively fixed throughout the year except as ice temporarily alters the stream channels.

Distribution Graphs for Watersheds With Stream-flow Records

12. Data available.- Rainfall and stream-flow records from which distribution graphs may be developed are plentiful in the Connecticut River Valley. There are approximately 100 rainfall stations in and

around the Connecticut River Watershed for which the United States Weather Bureau publishes daily records of precipitation and 16 stations for which hourly records are available. Stage and discharge hydrographs were obtained from the Water Resources Branch of the United States Geological Survey for 30 stations varying in drainage area from 12 to 797 square miles. Stage hydrographs for 25 stations were from continuous recording instruments and for the other five stations from staff or chain-gage readings.

13. Selection of storms.- After a careful investigation had been made of rainfall and run-off conditions for all New England storms of the past ten years, nine were selected from which distribution graphs were derived. Factors considered were:

a. Short durations (36 hours or less).

b. Isolation in respect to other periods of rainfall.

c. Freedom from snow cover.

d. Depth of rainfall (one inch or greater).

e. Run-off conditions favoring a high percentage of run-off.

14. Correction for artificial storage.- An investigation was made to determine the approximate volume of storage in storage reservoirs and run-of-river plants upstream from all stations for which distribution graphs were derived. In most cases the storage could be considered negligible, but on those streams where it had an appreciable effect upon the discharge hydrograph, the operation schedules of the reservoirs or run-of-river plants were obtained and the discharge hydrographs corrected accordingly.

15. Determination of rainfall.- The average rainfall for a storm over a given watershed was determined from the records at all rain gages in and around the watershed, and the rainfall was broken down into six-hour periods by inspection of the rainfall distribution at nearby hourly

recording stations.

16. Procedure for storms of 12 hours or less duration.- The unit rainfall period selected for the development of distribution graphs was 12 hours and the discharge period 6 hours. Distribution graphs were derived by the following procedure:

a. Subtract from the discharge hydrograph the ordinates of base flow and the ordinates of run-off resulting from any rainfall other than the 12-hour storm being used. This resulting hydrograph represents the net surface run-off from the storm.

b. Determine the volume of run-off of this hydrograph.

c. Determine the average ordinates of successive 6-hour discharge periods; express them as percentages of the volume computed in (b); and plot them against time as the abscissae.

17. Procedure for storms of more than 12 hours duration.- Distribution graphs were derived by the following procedure:

a. (Same as (a) above.)

b. Estimate the rainfall for 12-hour periods, and from judgment apply a run-off factor to each period of rainfall.

c. By trial and error find the distribution graph which will reproduce the net surface run-off hydrograph when applied to the estimated volumes of run-off from each period of rainfall.

The distribution graphs developed for the tributary watersheds are shown on Plate No. 12.

18. Discussion of results.- Distribution graphs were derived for 22 of the 30 stations for which data were available. Hydrographs for

the other eight stations were either incomplete or were affected by regulation of artificial storage for which operating data were not available. For several stations in the lower half of the Connecticut Watershed it was necessary, because of scarcity of data, to develop distribution graphs from one storm only, that of March 17-18, 1936. However, this storm was an excellent one for distribution-graph derivation on the type of drainage area for which it was used, and confidence is placed in the accuracy of the graphs derived from it. Excellent results were obtained in developing distribution graphs for all stations for which sufficient good discharge data were available. The agreements between graphs developed from May and November storms substantiate the contention that the unit hydrograph is a function not of surface cover, which may be subject to seasonal change, but primarily of the topographic features of a watershed. The general agreements between the several graphs for each respective watershed are closer than any hitherto published.

19. Reconstitution of past floods.- The distribution graphs developed were verified wherever possible by reconstituting from rainfall and snow-cover data the November 1927 and the March 1936 floods. Because run-off from the March 1936 flood was composed largely of melted snow in the northern half of the Connecticut River Basin, it was difficult to estimate with certainty the proper amounts of run-off from each rainfall period. However, this condition did not exist in the lower half of the basin for the March 1936 flood, or in any section of the watershed for the November 1927 flood.

Distribution Graphs for Watersheds

Without Stream-flow Records

20. General method.- As the unit hydrograph is chiefly a function of the topography of a watershed, a unit hydrograph for a drainage area without discharge records, but for which the topographic features are known, can be estimated, provided the relationships between unit hydrographs and topographic features are defined. To accomplish this, three delimiting elements of the unit hydrograph were selected, namely:

a. Rate of peak discharge.

b. Duration of hydrograph.

c. Time of occurrence of peak discharge after beginning of rainfall.

These three properties were plotted against various drainage-area characteristics such as slope of the area - elevation graph, drainage area, slope of stream bed, ratio of length of basin to width, number of major stems comprising a drainage system, depth of basin, and many others. Over a hundred such relations, or combinations of relations, were plotted, and it was found that three topographic features are predominant in their effect on the unit hydrograph. These are:

a. Drainage area.

b. Slope of area - elevation graph.

c. The stream pattern expressed as the number of major stems of a watershed.

These watershed characteristics are listed in Table 4 for each tributary watershed for which unit hydrographs were developed. The drainage area variable was reduced to a constant by transferring the unit hydrographs for the various watersheds by laws of hydraulic similitude to unit hydrographs for corresponding 10-square mile model drainage areas. The peak discharges of these model unit hydrographs were plotted against the durations and times of peaking of the model graphs and good relations were defined. The peak discharges of the model unit hydrographs were plotted against the slopes of the model area - elevation graphs with the number of main stems as parameters to form the relations that tied the unit-hydrograph properties to features of topography. In addition to these general relations, a good relation was found to exist between the prototype peak discharge and the rate of discharge 12 hours after the peak.

21. Determination of watershed slope factor.- A graph of drainage area versus elevation equalled or exceeded was prepared from the topographic map for each watershed. It may be obtained by planimetering all the contours of the watershed and plotting the measured areas against the contour elevations. As a practical expedient the following method was devised. A large celluloid sheet, subdivided into squares that represent one square mile on the United States Geological Survey quadrangle sheets, was placed on the topographic map of the watershed. The average elevation within each square was estimated and the number of squares accumulated in the order of their decreasing average elevations, from which the graph of drainage area versus elevation equalled or exceeded was plotted. The area between this graph and the minimum elevation of the watershed was planimetered, and the average slope of the graph, hereinafter termed the watershed slope factor, determined therefrom in feet per square mile. The area - elevation graphs for tributary watersheds are shown on Plate No. 14.

22. Definition of stream pattern.- The number of major stems of a watershed was determined by an inspection of the topographic map. A one-branch stream is defined as having no single tributary that drains more than 25 per cent of the total drainage area. A two-branch stream

is one having two major branches of approximately the same size draining at least 50 per cent of the total drainage area. A three-branch stream is classified similarly with the provision that the three branches drain at least 75% of the total drainage area. It is possible that the peak flow from one branch of a watershed may be so desynchronized by artificial storage or natural topographic conditions that the classification of the stream pattern would be altered. This was the case on the Sugar River, which, by inspection of topography, appeared to be a two-branch stream, but because of the effect of Sunapee Lake and other lakes on the same branch the peak discharge was a function chiefly of one stem. The classifications of tributary watersheds for which distribution graphs were derived are shown on Plate No. 13.

23. Pertinent principles of hydraulic similitude.- To transfer a unit hydrograph from the prototype to the model, the following relations were used:

$$q = Q/n^{5/2}$$

$$t = T/n^{.5}$$

$$h = H/n$$

$$a = A/n^2 = 10 \text{ sq. mi.}, \quad n = \sqrt{A/10}$$

$$s = nS$$

where

Q,q = discharge

T,t = Time

H,h = depth of rainfall

A,a = drainage area

S,s = watershed slope factor

1/n - scale ratio

(Capital letters signify prototype; small letters, model)

A model hydrograph reduced from a unit hydrograph using these relations would represent 1/n inches of run-off on the model. According to the unit-hydrograph theory, discharge ordinates vary directly with volume of run-off.

Therefore, to produce a unit hydrograph, or a hydrograph of one-inch depth of run-off on the model, the discharges must be multiplied by n, or the prototype discharges reduced by the relation, $q = Q/n^{3/2}$.

24. Determination of unit hydrograph elements for model watersheds.- A unit hydrograph for a model watershed is derived from a unit hydrograph for a natural area by the method shown in the following sample computation:

Actual watershed values:

$A = 200$ square miles

$S = 5.5$ feet per square mile

Stream pattern = 2 stems

Q_{12} = Peak discharge of unit hydrograph for 12-hour storm = 3200 c.f.s.

T_{T12} = Duration of unit hydrograph = 4.9 days

T_{R12} = Time of peaking = 20 hours

D = Duration of rainfall = 12 hours

Model values:

$a = 10$ square miles

$n = \sqrt{\frac{A}{a}} = \sqrt{\frac{200}{10}} = 4.46$

$s = nS = (4.46)(5.5) = 24.5$ feet per square mile

Stream pattern = 2 stems

$q = Q/n^{3/2} = 3200/4.46^{1.5} = 340$ c.f.s.

$t_t = T_{T12}/n^{0.5} = 4.9/4.46^{0.5} = 2.3$ days

$t_r = T_{R12}/n^{0.5} = 20/4.46^{0.5} = 9.4$ hours

$d = D/n^{0.5} = 12/4.46^{0.5} = 5.7$ hours

25. Adjustment of model watershed unit hydrographs to constant storm duration.- As may be seen in the preceding illustration, d, or storm duration on the model, varies for different watersheds dependent on size of natural area. Because of this it was necessary to adjust all

model unit hydrographs so that they would conform to the same storm duration, which was chosen as 12 hours. The adjustment of peak discharge was accomplished by constructing a curve of q versus d as follows:

Let Q_1 be the discharge 12 hours after the peak discharge on the prototype unit hydrograph, then $\frac{Q_{12} + Q_1}{2}$ would equal (very close approximation) the peak discharge of the unit hydrograph for a rainfall of 24 hours duration. Likewise if Q_2 equals the discharge 24 hours after the peak, $\frac{Q_{12} + Q_1 + Q_2}{3}$ would equal the peak of the unit hydrograph of a 36-hour storm duration. These peak discharges and their corresponding rainfall durations transferred to the model define the curve of q versus d. These curves for the unit hydrographs developed from discharge records are shown on Figure 1 of Plate No. 15. The values t_t and t_r were adjusted by adding a constant which varies inversely with d.

The adjustment of the model unit hydrograph, derived in the preceding paragraph, to one for a 12-hour storm duration is made as follows:

The topographic characteristics remain the same.

q_{12} is the value corresponding to d = 12 on the q vs. d graph = 297 c.f.s.

$t_{t12} = t_t + \frac{(12 - d)}{24} = 2.3 + .3 = 2.6$ days

$t_{r12} = t_r + (12 - d) = 9.4 + 6.3 = 15.7$ hours

d = 12 hours.

26. General relations.- The values of q_{12} were plotted against their respective values of s with the number of streams as a parameter, and they also were plotted against their respective t_{t12} values and t_{r12} values to form the relations shown on Plate No. 14. The prototype values of Q_{12} were plotted against the rates of discharge 12 hours after the peaks as shown on Figure 3 of Plate No. 15.

27. <u>Distribution graphs for watersheds without stream-flow records</u>.- To construct a distribution graph on a watershed without stream-flow records, it is first necessary to obtain A, S, and its stream pattern from a topographic map of the area. The three delimiting values of the unit hydrograph, Q_{12}, T_{T12} and T_{R12} may be determined mathematically by a reverse of the procedure described in the preceding paragraphs. However, the general relations have been prepared graphically and are shown on Figure 2 of Plate No. 15 from which these values can be obtained directly. From Figure 3 of Plate No. 15 the discharge rate 12 hours after the peak of the unit hydrograph may be obtained. Knowing the peak discharge rate, its time of occurrence after beginning of rain, the discharge rate 12 hours after the peak, and the duration of run-off, the unit hydrograph for a given drainage area may be constructed. From this unit hydrograph a distribution graph may readily be obtained. An example of the use of the general relations is shown on Figures 2, 3, and 4 of Plate No. 15.

28. <u>Discussion of results</u>.- The relations developed between unit hydrograph properties and drainage-area characteristics are fairly well defined by the data used although refinements and improvements are desired that were not made because of limitations in time and data. It is desired that the curves be better defined and substantiated by computed points developed from other watersheds. It is believed that a more exact expression may be found for evaluating the stream pattern of a basin and that a refinement may be introduced to distinguish between watersheds for which the area-elevation curves have the same average slope but markedly different shapes. A check on the dependability of the general relations developed will be made when sufficient discharge data is collected at reservoir sites to derive distribution graphs. Examination

of a few stage hydrographs that have been obtained at reservoir sites reveals no conflicts between them and the shapes of the distribution graphs estimated for those areas.

Distribution Graphs for Rainfall of Short Duration

29. General method.- Distribution graphs were needed for increments of rainfall with durations as short as one and one-half hours in order to determine the spillway-design floods from the spillway-design storm, which is described in the paragraphs following. The peak discharge accompanying a fixed volume of run-off on a given drainage area will vary inversely with the duration of rainfall from which the run-off is derived. Since it was not practicable, because of limitations of data, to determine directly distribution graphs for storms of shorter duration than 12 hours, the following method was devised by which the unit hydrographs for 12-hour storms were divided successively into unit hydrographs for six, three, and one and one-half hour storms and then converted into distribution graphs. The 12-hour unit hydrograph may be regarded as composed of two hydrographs of one-half inch of run-off each and resulting from two consecutive six-hour rainfalls of equal intensity. These two hydrographs would be identical in shape and may be obtained graphically as shown on Figure 4 of Plate No. 15. By construction $g_1 = g_1'$, $g_2 = g_2'$, etc. and the resulting hydrographs are identical, each representing one-half inch of run-off. From either of these hydrographs the six-hour unit hydrograph or the six-hour distribution graph may be computed. The six-hour unit hydrograph may be broken down similarly into the three-hour unit hydrograph, from which in turn may be obtained the one and one-half-hour unit hydrograph.

METHOD OF FLOOD ROUTING

30. General.- The effect of natural valley storage in modifying the peak discharge of floods is commonly known. An accurate evaluation of it is necessary in reconstituting past floods with incomplete stage records, in determining the resultant main stem hydrographs from hypothetical design floods on major tributaries, and in determining the modified hydrographs resulting from reservoired storage. The method used in flood routing was developed in the United States Engineer Office, Zanesville, Ohio, on the Muskingum Watershed Conservancy Project.

31. Basic principle.- This method is based upon the principle that the ratio of valley storage to a weighted flow determined from both inflow and outflow is constant throughout the entire range of stage for a given length of river valley, hereinafter termed a reach. The value of this ratio is dependent upon the physical shape of the valley within the reach. In equational form the principle may be stated:

$$K = \frac{T\left[.5(i_2 + i_1) - .5(d_2 + d_1)\right]}{X(i_2 - i_1) + (1.0 - X)(d_2 - d_1)}$$

Where K = Ratio of storage increment in reach in day-second feet to corresponding weighted flow increment in cubic feet per second,

T = Time unit of computation in days or fractions of a day.

X = Fraction of weighted flow increment that is derived from the inflow increment.

i_1, i_2, etc., = Total instantaneous inflow in cubic feet per second to a reach at beginnings of successive time units (T).

d_1, d_2, etc., = Instantaneous outflow in cubic feet per second from a reach at beginnings of successive time units (T).

The numerator of the equation is the storage increment, which is equal to the inflow minus the outflow in day-second-feet, while the denominator is the corresponding weighted flow increment in cubic feet per second. The significance of the weighted flow increment may be visualized more easily by rearranging it in the following form:

$$(d_2 - d_1) + X \left[(i_2 - i_1) - (d_2 - d_1) \right]$$

The first term is an index of the "prismatic" storage increment below the normal surface slope, and the second term is an index of the "wedge" storage increment produced by the changing slope during rising and falling stages.

32. Applicability of flood routing method to Connecticut River.- It is more difficult to determine accurately the effect of valley storage upon local inflows than upon the main stem inflow because the former do not traverse the entire length of the main river reach and therefore are not reflected in the stage at the upper end of the reach. Accordingly, the smaller their magnitude in proportion to the total flow through the reach the more accurate will be the flood-routing computations. The Connecticut River Basin is long and narrow, and as a result the proportion of drainage area tributary to any valley-storage reach selected is usually small compared with the drainage area subtended by the inflow station at the head of the reach. Owing to the comparatively small tributary inflows to each reach, there are no large wedges of backwater to alter the synchronized rise and fall of stage within the reaches. Consequently, a fair degree of accuracy may be expected from the application of the flood routing method to the Connecticut River.

33. Selection of reaches.- The flood routing method is basically the computation of the relation between discharge through a reach and valley storage within the reach. Since the river surface is continually being warped by the desynchronized contribution of tributary discharges and by the variation in rate of rise or fall of the main river discharge, it is evident that the accuracy of computation of the valley storage effect will vary inversely as the length of the individual reaches.

Generally the number of reaches is limited by the number of points along the river for which relations of stage to discharge have been determined. These may be computed, of course, but actual field measurements of discharge throughout the entire range of stage are much more accurate. For the purpose of flood routing, the Connecticut River was divided into seven reaches shown on Plate No. 16, extending from Dalton, New Hampshire, to Gildersleeve Island located 16.8 miles below Hartford, Connecticut. In selecting these reaches, especial attention was paid to the square miles of local drainage area subtended by the reaches, to the location of existing power dams, and to the location of all main-stem gaging stations of the United States Geological Survey. As a result, it was possible to compare the computed natural outflow hydrographs for the 1927 and 1936 floods at the lower end of each reach with the measured natural outflow hydrographs. In Table No. 5 are given pertinent data concerning these reaches.

34. Accuracy of relations of stage to discharge.- On the Connecticut River there are five well-rated gaging stations of the United States Geological Survey and several power dams, two of which were used as reach termini. Consequently, it was necessary to compute the stage-discharge relation at only one point, Hartford, in order to obtain the data necessary for flood routing. In all cases the river cross sections controlling the discharge at the reach termini are subject to relatively little scour and fill, and as a result the ratings of the stages do not change appreciably with time. The lower part of the Connecticut River is affected by the fluctuation of the tides during low water, but it is not noticeable during periods of flood flow when it might affect flood-routing computations.

35. Determination of Connecticut River K's.- The relationship (K) of valley storage to discharge within each reach was established by

plotting the peak discharges for three flood profiles against the valley storage beneath them, determined from measured valley cross sections which were taken approximately three and one-half miles apart along the main stem from Paper Rock below Middletown, Connecticut to McIndoes, Vermont. A total of 85 cross sections were measured, covering a distance of 249.0 miles. The locations of these sections were selected at representative points, where they would give the most correct evaluation of the natural storage of the valley. Cross sections, totalling 25 in number, were also taken on the main tributaries where the location of the gaging stations were several miles from the mouth and tributary valley storage was appreciable. In computing the volume of natural valley storage for any reach from measured valley cross sections, it was found that the storage below low-water flow was such a small percentage of the total storage at flood times that it could safely be neglected without affecting the results, except where existing dams provided appreciable pondage. In these cases, the volume of storage behind the dam at low-water flow was subtracted from the total storage when the relationship of discharge to valley storage was being determined.

36. The natural high-water profile for any flood is formed not only by the flood crest advancing down the valley, but by the addition of local inflow. In flood routing, each of these components is evaluated separately. Therefore, it was necessary to determine hypothetical profiles that would result from the flood crests advancing down the valley alone. These were obtained by adjusting the 1927 and 1936 high-water profile stages to the levels that would have been reached if the recorded outflow had entered the upper end of each reach. The hypothetical flood level at the upper end of the reach was determined from the rating curve. The stage differentials throughout the reach were varied directly as the distance from the lower end of the reach.

A third profile, at an intermediate flood stage was computed, and the valley storage beneath it measured to define more completely the K relation. The relations of valley storage to discharge are shown on Plate No. 17, from which it can be seen that K is approximately a constant for the entire range of flood stage in each reach.

37. Determination of tributary K's.- When there is more than one source of inflow (i.e., when tributaries enter the reach), it is probable that all inflow will not rise and fall synchronously, especially when modified by reservoir operation. In such cases valley storage will "warp" with variation in inflow, and a more accurate evaluation of its effect can be made by routing separately each inflow from its point of entry into the reach to the lower end of the reach. To accomplish this, it was necessary to determine individual K's for each point of inflow to the lower end of the reach. This was done by computing tributary K's from the gaging station to the mouth of the tributary by the use of measured cross sections and adding them to the main-stem K's from the mouth of the tributary to the lower end of the reach. In the Connecticut River Watershed, it is possible for a part of the tributary valley storage to act twice in reducing peak discharges for any one flood. This is owing to tributaries in the lower part of the watershed predischarging their peak flows before the peak flow of the main stem is reached at the confluence of the tributaries with the main stem. When this occurs, the natural tributary valley storage within the backwater effect of the main stem, acts first to reduce the peak discharge of the tributary and later to reduce the peak discharge of the main stem. The valley storage in the lower Farmington River acted in such a manner during the March 1936 flood.

38. Determination of X.- The X value was obtained and the ratio, K, was checked by plotting successive accumulations of the numerator in

the basic equation against the corresponding values of the denominator for the 1927, 1933 and 1936 floods, using X as the parameter. The curve approaching most nearly to a straight line for each flood satisfies most closely the equation, and therefore, determines the proper X for the reach. The ratio, K, is the mean slope of this curve, considering the accumulated numerator to be the abscissa. It was approximately in agreement with the K determined from measured cross sections.

39. <u>Comparative merits of inflow-minus-outflow computations and measured valley cross sections for the determination of K.</u>- It is desirable to determine K, the relation of valley storage to discharge, both from an accumulation of successive differentials between inflow and outflow and from storage-volume computations based upon field measurements of valley storage cross sections. The principal advantage of the former method is that it reflects the actual valley storage effect during floods of record. The attempt is made to eliminate the errors that arise from inaccuracy of the stage-discharge relations at both ends of the reach by adjustment of one or both rating curves. However, a balance for the entire flood period does not necessarily provide an accurately adjusted rating curve for its entire stage range, and therefore, the differentials between inflow and outflow may still be considerably in error. The greatest danger in use of the inflow-outflow method lies in the determination of the mean value of K for the stage range of a great flood from the mean value of K for the stage range of minor floods of record, because the mean slope of the storage-discherge graph for a small flood may be either greater or less than the mean slope for a great flood. This is illustrated by the valley storage relations for a typical reach shown on Plate No. 18. The valley cross section (Figure 1) was chosen as being typical of the lower reaches of the Connecticut River. In order to determine a typical stage-discharge

curve for the section (Figure 3), a constant slope of one foot per mile was assumed and the discharge computed from Manning's Formula:

$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$$

Where Q = discharge in cubic feet per second

A = area of cross section in square feet

R = hydraulic radius in feet

S = ratio of fall due to friction to length or reach

n = coefficient of roughness

An "n" value of .03 was used for computing the channel discharge, and a value of .075 for the overbank discharge. The stage-area curve (Figure 2), which is also a stage-storage curve, since a reach of unit length was taken, was combined with the stage-discharge curve to produce the discharge-storage relation as shown by Figure 4. The reliability of an extension of this last relation is dependent entirely upon the relative proportions of channel and overbank cross-sectional areas.

40. Limitations of storage volume computations.- Storage volume computations provide a definite physical measurement of the valley storage within a reach, if a sufficient number of cross sections are obtained, but it may not include the backwater storage on all of the minor and larger streams tributary to the reach. This was found to be true in five of the seven valley-storage reaches of the Connecticut River and the valley storage was increased as shown on Plate No. 17. Experience has shown in similar studies on the Ohio and Mississippi Rivers that the main stem valley storage may be increased from 5 to 20 per cent by this additional storage. Since high-water profiles are often obtainable where discharge records are not, this method will often provide the only means of solution.

41. Flood routing procedure.- In flood routing, i_1, i_2, and d_1,

are known, and it is necessary to solve for d_2. Solving the basic equation for d_2:

$$d_2 = - \frac{KX - .5T}{K - KX + .5T} i_2 + \frac{KX + .5T}{K - KX + .5T} i_1 + \frac{K - KX - .5T}{K - KX + .5T} d_1$$

which is in the form $d_2 = C_o i_2 + C_1 i_1 + C_2 d_1$

Where

$$C_o = - \frac{KX - .5T}{K - KX + .5T}, \quad C_1 = \frac{KX + .5T}{K - KX + .5T}, \quad C_2 = \frac{K - KX - .5T}{K - KX + .5T}$$

Graphs of C_o, C_1, and C_2, as abscissae with three series of K as ordinates (for T = 1/4, 1/2, and 1 day) and the probable range of X as the parameter were computed. From them, after K and X were determined, the coefficients were interpolated, and the flood routing carried on by tabular solution of the equation for successive d's. The flood-routing coefficients for each reach of the Connecticut River are given in Table 5.

42. Agreement between computed and actual Connecticut River hydrographs.- The hydrographs at the ends of the Connecticut River reaches were computed for the 1927 and 1936 floods by the routing method and were then compared, as shown on Plate No. 19, with the hydrographs determined from the stage records and the rating curves. The agreement obtained is well within allowable limits for application of the method to produce hypothetical floods and to determine reservoir effects. For the latter use, all computed modified hydrographs were adjusted by the discharge differentials between computed and actual natural hydrographs in order to make the modified hydrographs and the natural hydrographs of record comparable.

Application of Flood Routing Method to

Montague City - Thompsonville Reach

43. Description of Montague City - Thompsonville Reach.- This reach is composed of the Connecticut River between Montague City, Massachusetts, and Thompsonville, Connecticut; the Chicopee River from Bircham Bend to its mouth; and the Westfield River from Westfield to its mouth. Since Bircham Bend is located approximately 5.5 miles from the mouth of the Chicopee River and Westfield approximately 7.5 miles from the mouth of the Westfield River, the amount of tributary valley storage is appreciable. All inflow entering the reach at these points and the outflow at Thompsonville is gaged by well-rated gaging stations of the United States Geological Survey. All inflow to the reach not entering at these points is classified as "local inflow". The drainage areas of the various sources of inflow are:

Point	Drainage Area Sq. Mi.
Montague City	7,840
Bircham Bend	703
Westfield	497
Local	597
Thompsonville	9,637

44. Determination of K.- The floods of November 1927, March 1936, and a smaller computed flood were used for determining K for the reach. The volume of valley storage under each profile was determined from measured valley cross sections in the manner described in Paragraphs 35 and 36. The discharge-valley storage relation as determined by these floods is plotted on Plate No. 17. The individual K's were determined by computing separately the K for the tributary from the point of inflow

to the mouth and adding to this the main stem K from the mouth of the tributary to the lower end of the reach to obtain the total K. Following is a tabulation of the computation:

	Tributary K	Main Stem K	Total K
Chicopee	.07	.29	.36
Westfield	.18	.13	.31

45. Determination of X.- The floods of November 1927, April 1933, and March 1936 were used for determining X. To facilitate the computation of accumulative values of the numerator and denominator of the basic equation, it was written as follows:

$$K = \frac{T\left[.5(i_2 + i_1) - .5(d_2 + d_1)\right]}{x(i_2 - i_1) + (1.0 - X)(d_2 - d_1)} = \frac{T}{X}\frac{\left[.5(A) - .5(B)\right]}{(C) + (1.0 - X)D} = \frac{.5\,T\,(A - B)}{D + X\,(C - D)}$$

where

$A = i_2 + i_1$

$B = d_2 + d_1$

$C = i_2 - i_1$

$D = d_2 - d_1$

In Table 6 are shown the computations for the April 1933 flood from which the graphs for the determination of X were plotted. Inflow hydrographs at Montague City, Bircham Bend, and Westfield and the outflow hydrograph at Thompsonville were obtained from the United States Geological Survey. The hydrographs of inflow for the local drainage area were computed from rainfall by the "distribution graph" method. Since the total inflow and the outflow rates at the beginning and end of the flood are approximately the same, the total inflow volume should equal the total outflow volume. (The valley storage in the reach should be the same at the beginning and end of the flood). The total volume of inflow $= T\left[\frac{i_1 + i_2}{2} + \frac{i_2 + i_3}{2}, \text{etc.}\right]$ and likewise for the volume of outflow. In Table 6 at the bottom of Column (6) the inflow volume is shown to be 1969.5 day-second-feet, and at the bottom of Column (8), the outflow volume 1932.0 day-second-feet. Each inflow rate of Column (6) was corrected by the ratio $\frac{1932.0}{1969.5} = .981$ and the adjusted inflow entered in Column (7). Columns (9-12), inclusive, are computed from Columns (7 and 8) as their headings indicate. In Columns (13-23), inclusive, as their headings indicate, are given the computations for the abscissae and ordinates of the valley-storage curves plotted on Plate No. 18, Figure 5. From an inspection of Figure 5 it can be seen that the valley-storage curve, when X is zero, progressed a greater part of the time in a counter-clockwise direction. As X is increased, the distance between the rising and falling sections of each curve is decreased. The best value of X is 0.3, for which the valley-storage curve approaches most closely a straight line. However, there is little choice between X = 0.3 and X = 0.4, which indicates that the determination of X to closer than the nearest tenth is unwarranted.

46. Routing the 1936 flood.- The flood of March 1936 was routed by separate inflows to the lower end of the reach. Following is a table of routing coefficients interpolated from the curves shown on Plate No. 17 for each component of the reach.

Point of inflow	K	X	T	C_0	C_1	C_2	C
Montague City	.67	.3	.5	.065	.625	.31	1.00
Bircham Bend	.36	.3	.5	.285	.715	0	1.00
Westfield	.31	.3	.5	.33	.73	-.06	1.00
Local	0	-	-	-	-	-	-

The computations for routing the March 1936 flood are shown in Table 7 and the hydrographs on Plate No. 18. The agreement between the computed and actual outflow hydrographs is seen to be very close.

Hypothetical Floods.

47. Probable future floods.- The various types of future floods that may be expected are discussed in the main report. It is possible to constitute, by the use of the "distribution graph" and "flood routing" methods described in this section of the Appendix, the flood that would result on any tributary or at any point on the Connecticut River from any assumed occurrence of rainfall and/or melting of snow cover. It is evident that the number of combinations are unlimited. The expected frequencies of recurrence of peak flood discharges and flood volumes have been determined, are described in Paragraphs 5 and 6, and are shown on Plates Nos. 10 and 11. They define the most important indices of probable future floods, but give no indication of the relative efficiency with which various components of the total drainage area above any main stem station contribute to its maximum flood stages. In order to ascertain and analyze these values a number of hypothetical floods were constituted by the "distribution graph" and "flood routing" methods for a part of the range of the causative factors. The locations for which these floods were determined are White River Junction, Bellows Falls, Vernon, Montague City, Thompsonville, and Hartford. The floods are constituted for four types of storms on the drainage area above each of these locations, all of which were assumed to have an equilateral triangular distribution of intensity for a 3-day period, with an average depth of 4.5 inches. The only variable was the distribution of total rainfall over the watershed, which was varied as follows: storm No. 1 -

maximum depth of rainfall of 7 inches on the northernmost part of the watershed, minimum depth of 2 inches on the southernmost part of the watershed, and a straight line variation on the intervening area; storm No. 2 - maximum depth rainfall of 7 inches at the center of the watershed, minimum depth of rainfall of 2 inches at the northernmost and southernmost points in the watershed and a straight line variation for the intervening area; storm No. 3 - storm No. 1 rotated 180°; and storm No. 4 - uniform depth of rainfall of 4.5 inches on the entire watershed.

48. Determination of probable future flood hydrographs.- The Connecticut River Basin was divided into 8 component areas as shown on Plate No. 16. Unit hydrographs which had been prepared for several subdivisions of each component area were routed to the lower end of the component area and summated to form its unit hydrograph. Each of these in turn was routed through successive reaches of the natural valley storage of the Connecticut River to Hartford. The summations of the unit hydrographs from component areas routed to the selected locations of the hypothetical floods produced their unit hydrographs. The rainfall for each of the storms was converted entirely into run-off by applying it to each of the component unit graphs, and the resulting hydrographs were summated to obtain the hypothetical flood. Reference is made to Plates Nos. 20 - 25, inclusive, for these hydrographs, their components, and storm area - depth of rainfall relations.

49. Analysis of probable future flood hydrographs.- In the following table are shown the peak discharges for the hypothetical floods resulting from the four storm types.

Station	Peak Discharges - thousand c. f. s.			
	Storm 1	Storm 2	Storm 3	Storm 4
White River Junction	145.5	146.0	145.5	139.0
Bellows Falls	171.5	182.5	191.0	172.0
Vernon	185.0	206.5	212.5	189.0
Montague City	223.1	259.4	262.0	236.6
Thompsonville	259.2	302.5	296.8	272.4
Hartford	270.2	308.2	302.1	278.9

It can be seen that the variation and peak discharges at any location for the several storm types is not great and ranges from 5 to 17 per cent. The maximum flood is produced at each location by either storm No. 2 or storm No. 3 and the minimum flood by storm No. 1 at all points except White River Junction, where storm No. 4 produces the minimum. The method described in Paragraphs 56 to 58, inclusive, for obtaining individual reservoir effects has been adapted for use in showing the relative efficiency with which the various component areas contribute to the maximum flood stage below White River Junction. The flood contributing factor, C_W, is the percentage of the critical flood volume and peak discharge that the component area contributes to a flood crest at a main river station corrected for difference in unit volume of run-off. The efficiency factor $\frac{(M + N)}{2}$ is the contributing factor, C_W, divided by the ratio of the component drainage area to the total drainage area. Reference is made to Table 8 for these two factors for the hypothetical floods. An efficiency factor of 100 per cent represents merely the drainage area relation. It can be seen from the table that the range of efficiency factor is from 25 per cent to 145 per cent, and that the areas which contribute at the greatest efficiency to a main river flood are neither the ones farthest from nor nearest to the main river station, but the intermediate ones. Area I, above Fifteen Mile Falls, contributes to flood peaks in the lower river at greatly reduced efficiency regardless of storm type. The table invites inspection from the standpoint of the relative desirability of various parts of the watershed for location of flood controlling reservoirs. It shows that reservoirs in the Areas III - VII, inclusive, may be expected to produce reductions of Connecticut River peak discharges by considerably more than the usual drainage area relation. This is a characteristic of the Connecticut River Watershed which is seldom found to such a degree and may be attributed mostly to the elongated shape of the watershed.

50. Determination of maximum predicted floods.- It is within the realm of possibility that any maximum predicted flood stage or discharge may be exceeded. For purposes such as design of waterfront structures, dikes, and so forth, relatively safe maximums may be determined from a study of past occurrences. Such floods have been constituted only at Montague City, Springfield, and Hartford, in the lower Connecticut River area, where dikes are being considered. The peak discharges and flood-volume curves for these stations shown on Plate No. 11 were extended to frequencies of 1,000 years and these values taken as practical maximum predicted ones. Spring floods constituted by a run-off from melting snow cover of 10 cubic feet per second per square mile and by run-off from rainfall with an equilateral triangular distribution of intensity for 3 days and with sufficient volume to equal the maximum predicted when added to the run-off from melting snow cover would produce the maximum predicted peak discharges obtained from the frequency curves. Maximum predicted flood hydrographs are shown on Plate No. 26. Their limiting values in comparison with the maximum flood of record are tabulated below:

(Table on following page)

Station	Drainage Area sq. mi.	Maximum Predicted Flood: Volume of R.O.: in.	AF(ths)	Peak Discharge: c.f.s.	c.f.s./sq. mi.	Maximum Flood of Record: Peak Discharge: c.f.s.	c.f.s./sq. mi.	MP/MR
Montague City	7,840	8.0	3,343.0	239,000	36.90	247,000	31.5	1.17
Springfield	9,596	7.45	3,811.0	318,000	33.15	282,000	29.4	1.13
Hartford	10,643	7.20	4,070.0	318,000	30.20	280,000	26.7	1.13

The peak discharge of the maximum predicted flood at Hartford is not greater than at Springfield because the addition of local inflow is approximately counterbalanced by the alleviating effect of the intervening valley storage.

51. The demonstration flood.- The maximum predicted flood at Montague City, with a unit volume of 8 inches, was developed throughout the entire watershed and termed a "Demonstration Flood." It was used to show reservoir effects principally for a flood with no inequalities of contributing unit volumes from various tributaries. The hydrographs for this flood are shown on Plate No. 27. It should be realized that this flood is progressively less than the maximum predicted flood for drainage areas smaller than at Montague City and is greater than the maximum predicted flood for points in the Connecticut River downstream from Montague City.

Determination of Modified Discharges and Reductions in Stage by Reservoirs.

52. General description of method.- A study of the effect of reservoired storage in modifying the peak discharges of several floods was necessary in order to evaluate the flood-prevention benefits of the various reservoirs under consideration. Modified discharges were determined at the index points for all damage zones described in Section 2 of the Appendix and then were used to enter rating curves to obtain

modified stages. Modified tributary discharges were computed at index stations by estimated storage operations and by empirical formulae, taking into account natural volume of run-off from the reservoired drainage area and the probable volume that would be stored in the reservoir. Modified discharges at the Connecticut River index stations were computed by routing the modified inflow hydrographs through the natural valley storage of the river. Individual reservoir effects were computed by adaptations of the methods of determining their group effects.

53. Determination of modified discharges and stages on tributaries.- Where proposed flood control reservoirs were located in the lower part of tributary watersheds and in the vicinity of stream gaging stations, reductions of peak flood discharges were computed by estimating the entire modified hydrographs due to reservoired storage. At all other tributary index stations the modified peak discharges were computed by the following formula. It is an application of the laws of hydraulic similitude, from which it was found that the peak discharge varies as the 0.75 power of the drainage area for small watersheds having the same physical characteristics.

$$Q_m = \left[\frac{A - \frac{\sum(aLv)}{(V)}}{A}\right]^{0.75} Q_n = \left[1 - \frac{\sum(aLv)}{AV}\right]^{0.75} Q_n$$

where Q_m = Modified peak discharge in cubic feet per second.

Q_n = Natural peak discharge in cubic feet per second.

A = Drainage area at index point in square miles.

V = Flood volume at index point in inches.

$\sum(aLv) = a_1L_1 v_1 + a_2L_2v_2 + a_3L_3v_3$, etc., for all reservoirs above index points.

a_1, a_2, a_3, etc., = Drainage areas above dam sites (1, 2, 3, etc.,) in square miles.

v_1, v_2, v_3, etc., = Flood volumes at dam sites (1, 2, 3, etc.,) in inches.

L_1, L_2, L_3, etc. = $\frac{S_1}{v_1}, \frac{S_2}{v_2}, \frac{S_3}{v_3}$, etc., = Ratios of reservoir capacities to flood volumes at dam sites (1, 2, 3, etc.)

where s_1, s_2, s_3, etc., = Capacities of reservoirs in inches.

This method assumed that at the dam site the modified hydrograph is proportional to the natural hydrograph. It is considered that this assumption is well within the limits of error that result in applying a general formula. The $\frac{v}{V}$ ratio corrects the relation for difference in volume of run-off at the dam site from volume of run-off at the index point. L in the formula corrects for the inability to store the entire run-off at the dam site when the flood volume is greater than the reservoir capacity. When the flood volume is less than the reservoir capacity L should be eliminated from the formula. The natural tributary hydrographs as modified by the Comprehensive Plan of Reservoirs for the November 1927 and March 1936 floods, and for the "Demonstration Flood," are shown on Plates Nos. 6, 9, and 27 and the stage and discharge reductions are given in Table 9.

54. <u>Reduction of tributary peak discharges by individual reservoirs</u>.- The reduction of tributary peak discharges at index points by individual reservoirs was computed from the formulae in the following sections, which are applications of the formula described in the preceding paragraph.

55. Determination of modified discharges and stages on the Connecticut River.- The modified discharges in the Connecticut River were computed by routing all inflow hydrographs as modified by reservoired storage through the natural valley storage. The modified Connecticut River discharges and stages for the November 1927 and March 1936 and for the demonstration flood are shown on Plates No. 6, 9, and 27 and are given in Table 9.

56. Reduction of main stem peak discharges by individual reservoirs.- Whereas the storage in an individual reservoir produces a measurable and generally dependable reduction of peak discharge at tributary index points, its effect on a Connecticut River index point is dependent largely on its action as part of a group of reservoirs. It is possible to compute the modified Connecticut River hydrographs for each reservoir by flood routing, added one by one to form a system, but theresults would be misleading, for each reservoir alters the flood-reducing possibilities of the next one. A more practical and representative value was obtained through the determination of an average between an effective volume of storage index, M, and an effective peak discharge index, N.

57. Volume Index, M.- This index was derived from the March 1936 flood for which more complete data were available than for any other one of record, and from the "Demonstration Flood." All tributary hydrographs at gaging stations of the United States Geological Survey, where available, and otherwise from rainfall and melting snow-cover applied to distribution graphs, were routed individually through the natural valley

storage of the Connecticut River to the index points. Approximate dam site hydrographs, routed to the main river index points, were obtained by proportioning from the routed tributary hydrographs according to the estimated volumes of run-off at the dam sites and at the tributary gaging stations. The effective volume index was then computed by the following formula. The values described below are indicated on the hydrographs of the following sketch.

$$M = \frac{(Q_n - Q_m)\ wVA}{(Q_n)\ W_{va}}\ 100$$

where M = empirical flood reduction from storing of effective flood volume as a per cent of the drainage area relation.

Q_n = 1936 natural peak discharge at index point.

Q_m = 1936 modified peak discharge at index point within limits of possible reductions by a system of reservoirs.

W = empirical volume, a, b, d, c, e, between natural and modified hydrographs within time limits of effective storing. (Modified discharge at e was computed as

$$Q_m \left[1 - .75 \left(\frac{Q_n - Q_m}{Q_n} \right) \right]$$

a general value determined from a study of numerous modified hydrographs.)

w = volume, m, n, o, p, of routed natural discharge from a dam site within the time limits described above.

58. <u>Peak Discharge Index, N.</u>- Within the time limits described above there may be a considerable variation in the shape of the routed dam site hydrograph that, if stored, will produce varying efficiencies of peak discharge reduction, but the same value of M. To correct for this factor an effective peak discharge index, N, which allows for degree of coincidence with respect to time of the peak discharges of the natural hydrograph and of the routed dam site hydrograph, was computed by the following formula:

$$N = \frac{q\ V\ A}{Q_n v\ a} 100$$

where

N = empirical flood reduction from storing at instant of peak discharge at index point as a per cent of the drainage area relation.

q = discharge of routed dam site hydrograph at instant of peak discharge at index point.

It is noted that N would equal unity under the same conditions for which M would equal unity. Values of M, N, and their average derived from the March 1936 and the demonstration floods for all tributaries of the watershed are given in Tables 10-12 inclusive. The probable per cent reduction of peak discharge at each index point by each reservoir under consideration was computed from the formula:

$$C_w = \frac{(M + N)}{(\ 2\ )} \text{ avg. } \frac{aL}{A}$$

Values of $\frac{C_w}{L}$ for all reservoir sites under consideration are shown in Table 13.

$$C_{T-1} = 1 - \left[1 - \frac{a_1 \ L_1}{A} \right]^{0.75}$$

$$C_{T-2} = 1 - C_{T-1} - \left[1 - \frac{a_1 \ L_1 + a_2 \ L_2}{A} \right]^{0.75}$$

$$C_{T-3} = 1 - C_{T-1} - C_{T-2} - \left[1 - \frac{a_1 L_1 + a_2 L_2 + a_3 L_3}{A} \right]^{0.75}$$

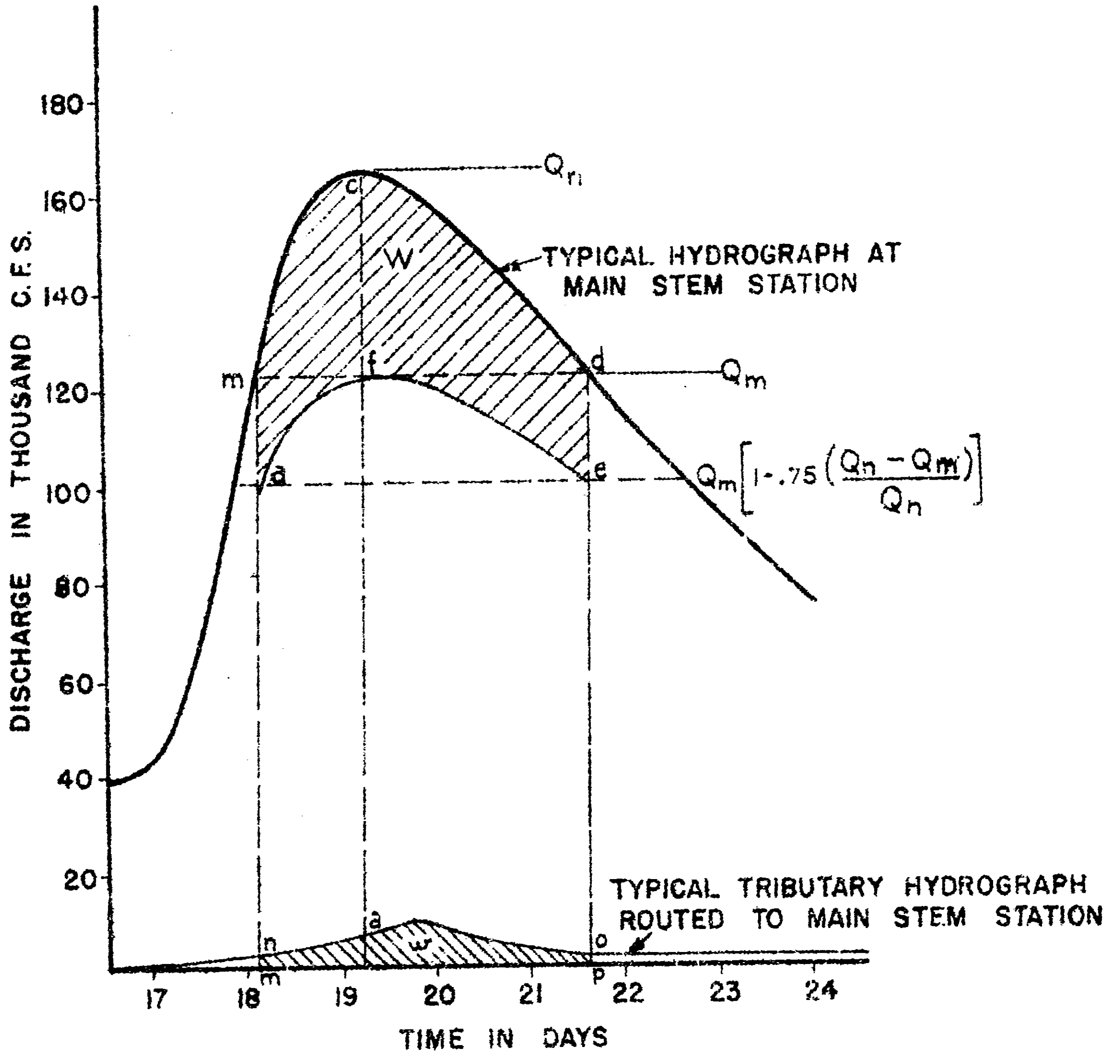
where

C_{T-1} = Ratio of reduction of natural peak discharge to natural peak discharge at tributary index points for first reservoir above index point in order of economic ranking.

C_{T-2} = ratio of reduction of natural peak discharge to natural peak discharge for second reservoir.

C_{T-3} = ratio for third reservoir.

Where several reservoirs were located above a tributary index point there was sufficient divergence in annual costs per acre-foot of storage to make possible the establishment of the correct order of economic ranking from this index alone. In obtaining the reductions by individual reservoirs as part of a group without priorities, each was computed as the first one, and this was used as a basis for proportioning the actual group reduction. When L, which is equal to $\frac{s}{V}$, is greater than 1.0, it is omitted. Stated otherwise, when the capacity of the reservoir is greater than the flood volume, C_T is a function only of the drainage areas. The $\frac{v}{V}$ ratios are omitted from the above equations because they are intended for use in determining reductions of normal floods of varying intensities for which the probable distribution of volume of run-off above the index point is a uniform value of v.



It is normally expected that the probable volume of run-off per square mile at any dam site above the index point is the same as at the index point. The $\frac{V}{v}$ ratio in the formula adjusts the M values derived from the 1936 flood to this normal expectancy. From a study of the above sketch it can be seen that, if the routed dam site hydrograph were proportional in shape to the hydrograph at the index points, and if $\frac{V}{v}$ equalled unity, then M would equal unity.

SPILLWAYS

59. General.- The factors considered in determining the required length of spillway at each dam site were the size and shape of the spillway-design flood; the reservoir pool elevation at the beginning of the flood; the conditions of the reservoir outlets; the storage available above the initial pool elevation and below the maximum surcharge, or gross head, on the spillway; the discharge characteristics of the spillway cross-section and approach; and the minimum allowable difference in elevation between the top of dam and the maximum spillway surcharge, hereinafter termed the "freeboard".

Meteorological Investigations From Date of Record

60. Maximum summer or fall storms.- The maximum possible flood during the summer or fall at any dam site in the Connecticut River Basin must result from the highest probable percentage of run-off applied to the maximum possible storm during these seasons. As a guide for determining this maximum storm, a study was made of the greatest storms of record in the Northeastern United States. The times of occurrences of these storms are as follows: October 3-4, 1869; July 13-14, 1897; October 8-9, 1903; July 19-23, 1919; Aug. 13-17, 1919; November 3-4, 1927; and September 16-17, 1932. Graphs showing drainage area versus mean depth of rainfall were determined for these storms, and are shown on Plate No. 29. The mean depth of rainfall as determined from the upper envelope of the graphs varies from 14.6 inches on 20 square miles to 11.3 inches on 500 square miles.

61. Maximum winter or spring storms and snow cover.- The maximum possible flood during the winter and spring at any dam site must result from the combination of the maximum possible spring storm with a run-off factor of 100%, accompanied by the run-off from the

maximum possible rate of melting of a large accumulation of snow cover. The maximum spring flood of record on most tributaries of the Connecticut River occurred during the period March 17-25, 1936, and was formed by run-off at a factor of 100% from rainfall that varied in mean depth from 3.1 to 5.4 inches and by run-off from melting snow that varied from 2.5 to 5.5 inches. The greatest recorded depth of rainfall in the area affected by the storm occurred at Pinkham Notch, New Hampshire, and amounted to 13.6 inches for the period March 16-22, inclusive, of which 10.8 inches occurred in three days, March 17-19, inclusive. The relation between drainage area and mean depth of rainfall for the latter period was determined and is shown on Plate No. 29 from which it can be seen that the mean depth of rainfall varied from 10.6 inches on 20 square miles to 8.6 inches on 500 square miles.

62. Relation of rainfall intensity to time.- Knowing that intensity of precipitation during any storm period is seldom uniform, the relation between maximum rainfall depth and duration was determined from data and studies of record for the northeastern United States. Rainfall intensity studies for this area have been made by others for durations varying from 5 minutes to 5 days. The results of these studies, when converted from intensity to depth of rainfall and combined, determine the graphs of duration versus rainfall depth with probable frequency of recurrence as parameters as shown on Plate No. 29. From these graphs it may be seen that the total depth of rainfall which will occur on the average once in one hundred years varies from 3.0 inches for the one-hour period of maximum intensity to 7.8 inches for the 72-hour period of maximum intensity. The intensity for the same storm for periods of less than one hour increases rapidly, and the total depth of rainfall amounts to 1.1 inches for the maximum ten-minute period. From a study of excessive

short-time precipitation records for ten recording rain gages it was determined that the maximum depth of rainfall for ten minutes amounted to 1.6 inches.

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*(1) Bernard, M.M., "Formulas for Rainfall Intensities for Long Duration." Am. Soc. Civil Eng., Trans., Vol. 96, p. 592, 1932.
(2) Miami Conservancy District, "Storm Rainfall of Eastern United States." Tech. Rep't., pt. 5, 1936.
(3) Yarnell, D.L., "Rainfall Intensity - Frequency Data," U. S. Dept. Agri., Misc. Pub. No. 204, 1935.

63. Time of most intense precipitation during the storm period.- The period of most intense precipitation may occur at any time during the duration of a storm. Continuous rainfall records of the U. S. Weather Bureau for the twenty most intense storms in the past thirty years at Providence, Rhode Island, many of which are shown on Plate No. 29, and of the November 1927 storm at several points in New England shown on Plate No. 4 indicated that the most intense precipitation would probably occur near the mid-point of the storm period.

Studies of United States Weather Bureau for Corps of Engineers

64. Relations of maximum rainfall depths to duration.- The maximum possible storms for which spillways should be designed have been determined in the past by increasing by a factor of safety the maximum storm of record in the geographical region of the proposed reservoir. In order to establish a more rational method of determining the maximum storm, the U. S. Weather Bureau, in cooperation with the Corps of Engineers, U. S. Army, made a study of the meteorological conditions and topographic features influencing the storms on the drainage areas above all proposed dam sites. Since the effect of these factors upon the relation of rainfall intensity to duration has not yet been completely analysed or successfully evaluated the relation of maximum rainfall depths to duration was determined in the Weather Bureau study from the

rainfall records of great storms in the area considered. A graph showing duration versus maximum rainfall depth in percentage of the maximum depth for one hour as determined by this study is shown on Plate No. 29. According to this graph, the maximum depths of rainfall for 6-hour and 12-hour durations are 230% and 280%, respectively, of the maximum depth for one hour. From this graph the depth of rainfall that occurs during the one-hour period of greatest average intensity and during the one-hour periods of succeedingly lesser intensity may be determined as per cent of the maximum depth for one hour and for a storm of any given duration may be expressed in per cent of the total depth of rainfall. For a storm of any given total depth of rainfall and duration the volume of rainfall occurring during any one-hour period may be expressed as the average rainfall intensity for that period and thus establish the relation of rainfall intensity to time for that storm. The relation of rainfall intensity to time for 1.0 inches of rainfall in 36 hours with maximum intensity at the mid point of the duration was determined in this manner and is shown on Plate No. 29.

65. Synopsis of air mass theory.- When the temperature of air of a certain moisture content is reduced below the saturation temperature, precipitation results. The reduction of temperature is determined to a large extent by the reduction of atmospheric pressure as the air rises to higher altitudes. Thus, the rate of rise determines the rate of moisture release. Moist air masses rise when they encounter rising ground, and also when they meet a colder, heavier mass of air. In the first case, a topographic "front" is established, and in the latter case a cold "front". An upward slope is created between the rising ground surface and the moving air mass and also between the cold underlying air mass, and the upward moving

warm air mass. Along these fronts precipitation may be expected in proportion to the moisture content of the air and the rate of rise of the warm air mass up the resultant topographic or cold air mass slope. When cold fronts are formed in the winter and spring, the moist air masses are rather stable and their rates of rise as influenced by the topographic or cold air mass slopes are relatively uniform but in the summer and fall the air masses are less stable because of the greater and more rapid change of temperature, and once caused to rise, they may move aloft almost vertically at high velocity. In the summer and fall the instability and violent and unpredictable movement of the air masses preclude any accurate or near-accurate calculation of slopes of air masses.

66. Maximum depths of summer or fall rainfall.- The maximum hourly rate of precipitation for a locality, as determined by this study, was based upon expected maximum conditions of vertical ascent, temperature, pressure, and thickness of the moisture-laden air mass. The rate of vertical ascent depends upon the topographic features and the conditions of the incoming moisture bearing air and the colder air mass it encounters. For the unstable summer conditions the maximum expected rate of ascent was based upon the rate of rise required to produce maximum known precipitations. The maximum moisture content of air is a function of the temperature and pressure, and this function is known. The thickness of the moisture-laden air mass and the moisture content at various altitudes was based upon the results of observations of cross-sections of air. The rate of precipitation as determined from the maximum conditions, were assumed to continue for one hour and thus determine the maximum depth of rainfall for a duration of one hour. Since the moisture content varies for air coming from several directions,

depending upon the proximity of moist tropical climes and intervening ridges of topography, the maximum depths of rainfall for a duration of one hour as determined by this study vary from 3.4 to 3.9 inches. Portions of the watershed subject to similar maximum meteorological conditions are shown on Plate No. 23.

67. Maximum depths of winter or spring rainfall.- During the winter and spring the fronts are rather stable and the rate of ascent may be determined from the assumed velocity of the incoming warm air mass and the slope of the topographic or cold air front. The lower temperatures prevailing during these seasons limit the moisture content of the air, and the resulting maximum rates of precipitation are lower than those for the summer and fall seasons. The maximum depths of rainfall for a duration of one hour to be expected during the winter and spring as determined by this study, vary from 0.5 to 0.8 inches. Portions of the watershed subject to similar maximum meteorological conditions during the winter and spring are shown on Plate No. 23.

68. Maximum rates of melting snow.- Run-off from winter storms may be augmented by the water from melting snow cover. The determination of its maximum amount is based upon the maximum depth of snow that may accumulate at each locality and the maximum rate of melting. The first of these is determined from a study of past records of snow fall and the latter from the day-degree accumulation of temperature above freezing. The resulting estimated maximum rate of melting of snow cover is 1.2 inches of water per day for that portion of the watershed in the upper halves of New Hampshire and Vermont, and 1.4 inches per day elsewhere.

Comparison of Results -

Weather Bureau and Investigations from Data of Record

69. Relations of rainfall depth to duration.- The relations of rainfall depth to duration were determined from rainfall records, and since all the recent studies of this nature were made from approximately the same records, the resulting relations should be in close accord. For purposes of comparison the relations of rainfall depth to duration were plotted as time versus percentage of the depth for one hour, and are shown on Plate No. 29. From these graphs it is seen that the total depth of rainfall for the 12-hour period of maximum intensity is 280 per cent of the maximum depth for one hour, as determined from the Weather Bureau study, and 190 per cent from the other studies. This difference must result from the data used in the studies and the individual treatment of these data.

70. Maximum summer or fall storms.- The greatest summer storm of past record in the area considered occurred in more than one day and less than two days and therefore was estimated to occur in 36 hours, and varied in mean depth from 14.6 inches on 20 square miles to 11.3 inches on 500 square miles. The maximum summer or fall storms, as determined by the Weather Bureau study are shown as graphs of duration versus accumulated mean depth of rainfall on Plate No. 29. From these graphs it is seen that the maximum total depth of rainfall expected to occur in 36 hours varies from 13.2 to 11.3 inches on various portions of the watershed according to the topographic effect upon the air masses. The relation of depth of rainfall to size of drainage area was not determined in the Weather Bureau study, while lack of sufficient data precluded any evaluation of the topographic effect by a study of the storms of record. Assuming the maximum depths of rainfall to be expected in 36 hours from the Weather Bureau study to apply for drain-

age areas of 50 square miles, graphs of drainage area versus mean depth of rainfall for various portions of the watershed were determined from the relation of area to depth established by the greatest storm of record in northeastern United States and are shown on Plate No. 29. From these graphs it is seen that the maximum mean depth of rainfall expected to occur in 36 hours on a drainage area of 20 square miles varies from 11.7 to 13.7 inches and on a drainage area of 500 square miles varies from 9.4 to 11.0 inches.

71. Maximum winter or spring storms and snow cover.- The maximum winter or spring storms as determined by the Weather Bureau study are shown as graphs of duration versus accumulated mean depth of rainfall on Plate No. 29. From these graphs it is seen that the maximum total depth of rainfall expected to occur in 36 hours varies from 4.5 to 7.0 inches on various portions of the watershed according to the topographic effects upon the air masses. The maximum expected run-off from melting snow cover varies from 1.2 to 1.4 inches on different portions of the watershed and must be added to the run-off from rainfall. The greatest winter or spring storm in the area considered occurred during the period March 16-22, 1936. The mean depth of rainfall that occurred during the 72-hour period of maximum intensity varied from 10.7 inches on 20 square miles to 8.6 inches on 500 square miles. The center of this storm occurred in the mountainous region of New Hampshire, just east of the Connecticut River Watershed, and the rugged topography in this region undoubtedly had marked effect upon the rate and volume of precipitation. The maximum mean depth of rainfall on any tributary of the Connecticut River during the period March 16-22, inclusive, 1936, occurred on the Chicopee River at Bircham Bend, and amounted to 5.4 inches, while the maximum run-off from melting snow cover occurred on the Ottauquechee River at North

Hartland and amounted to 5.5 inches. These values are well under the maximum to be expected, as determined by the Weather Bureau study. Graphs of drainage area versus mean depths of rainfall for various portions of the watershed were determined, as for the summer storms, from the maximum depths of rainfall in 36 hours from the Weather Bureau study and the relation of area to depth from the greatest storm of record, and are shown on Plate No. 29. From these graphs the mean depth of rainfall expected to occur in 36 hours on a drainage area of 20 square miles varies from 4.7 to 7.2 inches and on a drainage area of 500 square miles varies from 3.8 to 5.8 inches.

Adopted Design Storms and Resulting Floods

72. Adopted summer or fall storms.- The total depths of rainfall during the summer or fall spillway-design storms adopted for this report are shown as graphs of drainage area versus depth of rainfall on Plate No. 29. These graphs were determined by increasing by 50% the relations of drainage area to mean depths of rainfall for the maximum 36 hour summer storms from the Weather Bureau study. Due to the meteorological conditions and topographic features influencing the storms on various portions of the watershed, the mean depth of rainfall during the summer or fall spillway-design storm on a drainage area of 20 square miles varies from 17.6 to 20.6 inches and on 500 square miles varies from 14.1 to 16.5 inches. The graphs of drainage area versus mean depth of rainfall for the summer or fall spillway-design storm are identified by letters and govern those portions of the watershed similarly identified and shown on Plate No. 28. The relation of rainfall intensity to time, adopted for these storms was determined from the Weather Bureau study and is shown on Plate No. 29 as a graph of rainfall intensity versus time for 1.0 inches of rainfall in 36 hours. The rain graph determined from these relations for

a typical spillway design flood with a total depth of rainfall of 18.3 inches is shown on Plate No. 29. The maximum rainfall intensity from this graph is 12.6 inches per hour and the average rate for the 1-1/2 hour period of maximum intensity is 4.4 inches per hour.

73. Adopted winter or spring storms.- The total depths of rainfall during the winter or spring spillway-design storms adopted for this report are shown as graphs of drainage area versus mean depth of rainfall on Plate No. 29. These graphs were determined by increasing by 50% the relation of drainage area to mean depths of rainfall for the maximum 36 hour winter storms from the Weather Bureau study. As determined by these graphs the mean depth of rainfall on a drainage area of 20 square miles varies from 7.0 to 10.9 inches and on 500 square miles varies from 5.7 to 8.3 inches. The graphs are identified by letters and govern those portions of the watershed similarly identified and shown on Plate No. 29. The same relation of rainfall intensity to time was used for this storm as in the case of the summer storm. The adopted run-off from melting snow was determined by increasing by 50% the maximum rates as evaluated by the Weather Bureau study. The resulting rates of run-off from melting snow amounted to 1.8 and 2.1 inches per day and were assumed to run off at a constant rate.

74. Spillway-design floods.- Having determined the mean depth of rainfall and distribution of the spillway-design storm the resulting flood hydrograph at each dam site was computed from its distribution graph using time increments sufficiently small to evaluate the effect of rapidly changing storm intensities. In the summer and fall seasons, due to absorption and evaporation, the rainfall that runs off will be less than 100%. The per cent of run-off will depend upon ground and atmospheric conditions and the intensity of rainfall. Since the spillway-design is based upon the worst possible meteorological conditions, a run-off factor of 80% was used in computing the design flood hydrographs

for the summer and fall seasons. A typical flood hydrograph from the maximum summer or fall spillway-design storm is shown on Plate No. 30. In the winter and spring floods, the storm rainfall was considered to run off at a factor of 100%. The hydrograph from melting snow-cover was obtained by assuming a constant rate of run-off, and the total flood hydrograph at the dam site was obtained by adding the hydrograph from melting snow and the hydrograph from the winter storm. A typical flood hydrograph from the maximum winter or spring spillway-design storm is shown on Plate No. 30. The spillway surcharge storage at most of the proposed reservoirs is small and the maximum spillway discharge depends upon an accurate determination of the design-flood peak discharge. To do this accurately requires not only that the maximum rate of precipitation be known but also that the distribution graphs be accurately determined. The derivation of these distribution graphs is treated completely in Paragraphs 20 to 29, inclusive. In Table 14 are shown the peak discharge, flood volume, and flood duration for the summer or fall and winter or spring spillway-design floods at each dam site and on Plate No. 30 the peak discharge of the governing flood hydrograph at each dam site and also the estimated maximum discharge of record are plotted versus drainage area. The former are generally several times the latter.

Types of Spillways and Their Discharge Characteristics

75. Types of spillways.- The types of structures studied for this report are ogee, saddle, side-channel, and morning-glory spillways. Gated spillways and spillways with flashboards have not been considered. Possible submergence of the spillway was eliminated in all cases except in the morning glory type by adequate slope and discharge channel below the spillway. In the design of a spillway structure, it is desirous

to pass a given discharge using the most economical spillway section possible. The physical characteristics of the spillway structure and approach channel such as height and shape of the control, length and material of the approach channel, etc., each have a marked effect upon the ability of the structure to pass the discharge. A detailed evaluation of these factors is, therefore, an economic necessity and they are treated in the following. For this report, to care for factors which could not be evaluated at this time, safety factors of 10% were applied to all discharge coefficients except for those structures over 100 feet in height where a 5 per cent factor was used. The adopted spillway discharge coefficient at each dam site is given in Table 14.

76. Bibliography.- A bibliography of papers on theory and experimental data related to spillway discharge characteristics that are referred to in subsequent paragraphs are tabulated below:

1. Ogee Spillway.
   - *1 Bazin, H., "Annales des ponts et chaussees." - Oct. 1888
   - *2 Horton, R. E., "Weir Experiments, Coefficients, and Formulas", U. S. Geol. Survey, Water Supply Paper 200, 1907.
   - *2 King, H. W., "Handbook of Hydraulics". - 1929
   - *2 Russell, G. E., "Textbook on Hydraulics" - 1925
   - *3 Schroder and Turner. "Precise Weir Measurements" - Trans. A.S.C.E., Volume 93, 1929, Pg. 999.
   - *4 Cline, C. G., "Discharge Formula and Tables for Sharp-Crested Suppressed Weirs" - Trans. A.S.C.E. Volume 100, 1935, Pg. 296.
   - *5 Creager, W. P., "Engineering for Masonry Dams" 1929
   - *6 Dillman, O., "Untersuchungen an Ueberfallen" - Mitteilungen des Hydraulischen. Munich 1933
   - *7 Rouse, H. & Reid, L., "Model Research on Spillway Crests". Civil Engr., January 1935.
   - *8 Nagler, F. & Davis, A., "Experiments on Discharge over Spillways and Models, Keokik Dam." Proceed. A.S.C.E. February 1929.
2. Saddle Spillway.
   - *9 Bakhmeteff, B., "Hydraulics of Open Channels", 1932, Page 35.
   - *9 Thomas, H., "The Hydraulics of Flood Movements in Rivers." 1937.
3. Side-Channel Spillway.
   - *10 Hinds, J., "Side-Channel Spillways: Hydraulic Theory, Economic Factors, and Experimental Determination of Losses." Proceed. A.S.C.E. Sept. 1925.
   - *10 "Dams and Control Works." Dept. of Interior Publication. 1929.

4. Morning-Glory Spillway.
*11 Kurtz, F. & Jaenichen, P., "Hydraulic Design Analysis Pleasant Hill Dam Analysis of Design. 1935
*11 Kurtz, F., "Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam, and Hydraulic Tests on Working Models." Trans. A.S.C.E. Volume 88, 1925, Page 1

77. Ogee spillway.- Standard practice is to base the curve of the spillway on the profile of the fully ventilated nappe of water flowing over a sharp-crested weir. This practice is founded on the theory that the presence of a solid structure below the nappe, and in immediate contact with it, will not appreciably affect the course of the freely falling particles of water; that is, neither will the nappe tend to spring loose from the spillway face, nor will it exert pressure upon it. This is not essentially true as the presence of the solid structure below the nappe immediately reduces the status of flow to that of an unventilated condition with the resulting discharge exerting slight positive pressures against the structure. Since negative pressures are structurally undesirable, design by this method produces a small desired factor of safety. The spillway profile upstream from the centerline of the crest designated as the "nose" is simulated from experiments on aerated sharp-crested weir jet profiles as derived by Bazin, *1, the results of which are shown on Plate No. 31, Figure 1. The spillway face or downstream section is derived from a mathematical continuation of the Bazin jet. The maximum spillway surcharge used in computing the profile is designated as the "design head." The basic theoretical expression for the flow over a spillway is given as:

$$Q = CLH^{3/2}$$

where Q = theoretical discharge in c.f.s.
L = effective length of crest,
H = measured head in feet, and

C = the coefficient of discharge which depends upon the shape of the crest, velocity of approach, etc.

Many attempts have been made to experimentally evaluate the maximum value of the coefficient "C" by use of the sharp-crested weir, *2. Each experimenter gave conclusions as generalized formula which differ from those of other experimenters, but each is supported by experimental data over a definite range of head and height of weir. Perhaps the only set of experiments approaching completeness is that of Schroder and Turner, *3, produced in 1918 at Cornell University and shown on Plate No. 31, Fig. 2. A mathematical analysis of all existing data by C. G. Cline, *4, resulted in the acceptance of the Schroder and Turner results as a basis for his formula which is in closer agreement with the total range of data than any of the standard formulas by other experimenters. A comparison of the most prominent data, namely that of Bazin, Francis, Rehbock, and Schroder and Turner, is shown on Plate No. 31, Fig. 3.

78. Measurement of head.- The measurement of the head in sharp-crested weir experiments is taken as the vertical distance from weir crest to water surface at a point sufficiently remote from the weir to avoid the surface curve, which is at least two and one half times the head upstream from the crest. The measurement of the head on a spillway is the difference in elevation between the crest of the spillway and the energy gradient at a point corresponding to that at which the head is measured above the weir. An inspection of the Bazin curve for a vertical-faced weir shows that the variation between the weir crest and the high point of the under side of the nappe is 0.11 times the weir head and thus the head on a dam crest would be .89 times the head on the corresponding weir crest, plus the velocity head, or

$$H_d = .89H_w + V_w^2/2g$$

Conversion of the head at the weir crest in the Schroder and Turner experiments to head at spillway crest and substitution in the discharge formula will result in new values of the coefficient "C" as shown on Plate No. 31, Fig. 4. Coefficients ranging from 3.3 to 4.4 for the sharp-crested weir fall between 3.94 and 4.20 when converted to spillway coefficients. It is believed that an exact experimental analysis will point to a single value for "C" for the design head. The assumption that 0.110H is the difference in elevation between weir and dam crest for low heights of structure involves a small percentage of error as the limits of flow lines approaching the crests of various heights of structures simulate Bazin's curves d, e, f, etc., Plate No. 31, Fig. 1, in which the factor becomes 0.089H, 0.061H, 0.040H, etc., respectively.

79. <u>Coefficient, "C", at the design head</u>.- The single maximum value of "C" at the design head for all heights of structures has been accepted by W. P. Creager, *5, who derived C = 3.94 from data in Horton's paper. Experiments by Dillmann, *6, in Munich made in 1933 found this value to be about 4.04. Rouse and Reid, *7, in experiments at the Massachusetts Institute of Technology found this value to be 4.01. A value of 4.05 for the weir crest was brought to 4.02 in one case, showing that a small variation in discharge may be expected with the filling of the under-nap with masonry. This may be accounted for by the difference in pressure due to variation in surface tension and viscosity. A value of 4.01 has been accepted for this report. A study of discharge measurements on the actual structure and model of the Keokik, *8, shows that discharge measurements follow the 4.01 "C" value for the design head. Any increase in the crest section, over that shown by Bazin, with the flow adhering to the crest will result in

positive pressures. In general it may be said that positive pressures in the region of the crest are accompanied by a decrease in the discharge coefficient whereas a negative pressure signifies that the coefficient is increased. The need for maintaining the theoretically perfect nose is, therefore, apparent. As stated by Rouse and Reid, "A hydrodynamical approach to the problem of curving flow, such as that over weir crests, shows that conditions at any point in the flow are dependent upon those directly upstream. It has been demonstrated by Bazin and other investigators that any change in the crest of a weir will result in a change in the nappe profile". For any modification of the nose the coefficient "C" will vary from 4.01 and the degree of change will need to be determined by model study.

80. Coefficient, "C", at heads other than design head.- For a spillway designed for a given head, the 4.01 coefficient may be had only for that head. A smaller head on the crest would have a nappe lying within the spillway profile, and, therefore, positive pressures with accompanying decreases in the discharge coefficient. Experimental values for the full range of coefficients are given on Plate No. 31, Fig. 5.

81. Allowance for velocity head and friction loss in approach channels.- For this report, the definition of surcharge on a dam is the difference in elevation between the crest and the still pond. As previously mentioned, these coefficients apply to the head at a point just upstream from the crest and yet so remote from the dam as to avoid effects of the surface curve and H = depth of water + velocity of head. Where the approach channel was of sufficient length to entail friction loss, water surface curves were computed to still pond elevation to determine the surcharge. This gave a corresponding decrease in the

full range of discharge coefficients.

82. Saddle Spillway.- The saddle spillway is a broad-crested weir with a downstream slope, the grade of the slope determining the location of the control. Discharge at the control is critical discharge and may be computed by

$$Q = a\sqrt{a} / b \sqrt{g} \quad *9, \text{ where}$$

Q = discharge in c.f.s.
a = area of cross section in square feet.
b = width of section at water surface in feet.
g = acceleration due to gravity in feet per second per second.

Should the control be removed downstream from the spillway crest the surcharge is computed from the control by backwater computations. The head loss due to entrance is evaluated by adding 5 per cent of the velocity head at the crest to the head at the crest. The "C" coefficient in the formula $Q = CLH^{3/2}$ for critical flow is 3.087.

83. Side-channel spillway.- There is little precedent in side-channel spillway design other than the works of the Bureau of Reclamation, *10. A theoretical analysis of their design methods published in the Proceedings of the American Society of Civil Engineering, in September 1925, by Julian Hinds, is the basis of design for this report. Mr. Hind's analysis is predicated on the hypotheses that the energy of flow over the spillway-crest is entirely lost in the spillway channel and that the flow moves off at right angles to the direction of spillway inflow due to slope acquired momentum. The spillway may be divided into three sections as follows: the ogee spillway crest, the outflow channel above the control, and the outflow channel below the control. The ogee spillway section is designed as an ogee dam to the submergence line; discharge characteristics of this section are as previously described. The control or point of critical flow may be located at or downstream from the end of the ogee section. The section above the control is

designed to sufficiently remove the flow without crest submergence. The section below the control is designed with a slope sufficient to discharge the flow at a depth below critical.

84. Morning-Glory spillway.- The morning-glory or shaft spillway structure is a combined spillway and conduit, *11. The morning-glory section is a modified ogee section, the crest being circular in form. The discharge may be treated as for the ogee dam to the submergence limit with reductions in the crest length due to training piers, which may be evaluated at 4.5 per cent of the head for each contraction. The tunnel system is designed to pass the required discharge at the design head with the crest submerged.

(Report continued on following page)

Determination of Spillway Sizes

85. Initial pool elevation.- Since the streams in the Connecticut River Basin are susceptible to additional development in the future for power and other conservation, there is a possibility that, at some time within the life of each proposed flood-control reservoir, its present intended use for flood control may be modified in favor of usage for increasing the low-water flow for benefit of power or other conservation uses. Under such conditions, the reservoir might be filled to spillway crest at the time of the spillway-design flood. The possibility is far more remote for a reservoir with automatic outlets and no gate than it is for one with gate-controlled outlets. Therefore, wherever automatic outlets are used, the reservoir pool elevation at the beginning of the flood is considered to be at the bottom of the outlet, and for all proposed reservoirs with gate-controlled outlets, at the spillway crest.

86. Conditions of outlets.- It is assumed that outlet gates may be frozen in the closed position or rendered inoperable and the automatic outlets may be clogged with debris or closed in some other manner at the beginning of the spillway-design flood. This criterion for automatic outlets does not affect the criterion of Paragraph 85, concerning initial pool elevation, the reasoning being that although the automatic outlets are clogged with debris, so as to render their discharge capacity ineffective during the spillway-design flood, the clear opening will be sufficient to keep the reservoir empty during periods of normal stream flow.

87. General procedure of determining size of spillway.- The type and size of spillway desired at each site is that which will pass the spillway-design flood without endangering the safety of the dam structure and result in the least total cost of reservoir. For any given maximum surcharge on the spillway, there is a corresponding length of spillway

that will meet the first requirements. Also, there is a definite maximum spillway surcharge with its corresponding spillway length that will produce the least total cost of reservoir. The first step in the solution was to evaluate the spillway surcharge versus length for each site. As a maximum spillway surcharge increases the length decreases. Since the elevation of top of dam is equal to the sum of the elevation of spillway crest, which is considered fixed, the maximum surcharge on the spillway, and the design freeboard, which is also fixed as described later, the elevation of top of dam will vary directly as the maximum surcharge and, therefore, the cost of dam structure proper will vary directly as the spillway surcharge. Since the cost of spillway varies directly with its length, which varies inversely with the maximum surcharge, the cost of spillway will vary inversely with the maximum surcharge. Accordingly, the most economical combination is found by evaluating the total cost of reservoir versus surcharge and selecting the point of minimum cost.

33. Method of routing floods over spillways.- Given the hydrograph of inflow to the reservoir, the problem is essentially to determine the resultant pool elevations and outflows when the spillway is in operation. The method used in the solution of this problem was derived and applied as follows: For a given time increment the spillway discharge is equal to the inflow measured at the upper limit of the pool minus the increment of gross capacity withheld as surcharge storage. This equation is solved graphically through successive time increments of equal length. By making the approximations that one cubic foot per second for one day is equal to two acre-feet and that the mean rate of flow for the time increment is equal to the average of the rates of flow at the beginning and end of the time increment, the following equations can be written:

$$D_{1-2} = I_{1-2} - s_{1-2} \text{ ---------------------------(1)}$$

$$D_{1-2} = t(d_1 + d_2)$$

$$I_{1-2} = \frac{2^t(i_1 + i_2)}{2} = t\ (i_1 + i_2)$$

$$s_{1-2} = S_2 - S_1$$

$$t\ (d_1 + d_2) = I_{1-2} - (S_2 - S_1)$$

$$S_1 + I_{1-2} = td_1 + td_2 + S_2 \text{ -----------------(2)}$$

in which D_{1-2} = volume of spillway discharge in acre-feet.

d_1 and d_2 = spillway discharge rates in cubic feet per second at beginning and end of time increment.

I_{1-2} = volume of inflow to reservoir in acre feet.

i_1 and i_2 = rates of inflow to reservoir in cubic feet per second at beginning and end of time increment.

s_{1-2} = increment of gross capacity in acre-feet.

S_1 and S_2 = gross capacities in acre-feet at beginning and end of time increment.

t = time increment in factions of a day.

In equation (2) d_2 and S_2 alone are unknown. Their graphical solution is dependent upon the fact that they are both functions of pool elevations. Two curves of pool elevation versus td (volume of discharge in acre-feet during 1/2 the unit period, t) symmetrical with respect to the pool elevation axis, and with scales the same as those of the pool elevation versus gross capacity curves, are drawn on a sheet of transparent material. Reference is made to Plate No. 30 on which is indicated the solution for two successive time increments while the pool elevation is increasing, and for two successive time increments while the pool elevation is decreasing. The sheet of transparent material is superimposed upon the capacity curve of the reservoir, and continually kept oriented with respect to pool elevation. For the time increment (1-2) the volume (I_{1-2}) is laid off to the right of the point on the gross capacity curve with the abscissa (S_1) and the t d curves moved until the t d (right) curve passes through the point with the abscissa ($S_1 + I_{1-2}$). The pool elevation at the end of the time increment is found at the intersection of the t d (left) curve and the gross capacity curve. The solution obtained thusly follows through the equation in the order in which it is written. In applying this method it was found advantageous to scratch the t d curve on thin, transparent cellulose sheets to which wooden straight edges were stapled parallel to the t d axis. Then, after the two sheets are oriented with respect to elevation and a weighted T-square is placed against the straight-edge, successive steps of surcharge and discharge can be rapidly computed. Also, (t d) curves for several tentative sizes of spillways to be tested with the design criteria can be placed on the same cellulose overlay and the computations repeated for each size of spillway.

89. <u>Determination of maximum surcharge versus spillway length relations</u>.- The spillway-design flood, as limited by the condition of the reservoir described above, was routed as an inflow flood through the reservoir and over the spillway crest by 1/16 day time increments for various lengths of spillway to determine the maximum surcharge and discharge. The adopted surcharges and corresponding spillway discharges are given in Table 14. The effect of the natural valley storage within the reservoir was not considered because it appeared in preliminary investigations to be negligible. At Victory, Priest Pond, and Birch Hill dam sites the spring spillway-design flood produced the greatest length of spillway. At all other dam sites, the fall spillway-design storm produced the greatest length of spillway. In general it was found that surcharge storage is negligible compared to the flood volume, and,

therefore, it reduced the peak discharge of the design flood but little in those cases where the proposed reservoir is gate-controlled. Consequently, prime importance is attached to the accuracy of the computed values of peak discharge for the design flood.

Freeboard

90. Design freeboard.- The design freeboard as used herein is the differential in elevation between top of dam and water surface of maximum surcharge. The value may be as low as 1.0 foot for masonry dams, if proper provisions are made in the anchor design, and for earth dams should be a function of the fetch exposure to wave action on the upstream face of dam and the velocity head of the maximum expected waves. The freeboard was computed as 3/4 of the wave height plus the velocity head of wave velocity taken from the following formulae:

$h + 1.5 \sqrt{F + 2.5} - \sqrt{F}$ (Stevenson's Formula) and $V = 5 + 2h$ in which h = wave height, F = the fetch in nautical miles and V = the wave velocity. The theoretical freeboard at each dam site computed from this method is given Table 14. It varied from 3.8 to 4.7 feet. For the purpose of this report, the freeboard for earth dams was taken as 5.0 feet in all cases except Lower Naukeag where 4.0 feet was used.

Outlets

91. Basic factors.- The factors considered in determining the type and size of outlets were:

(a) The size flood that could be accommodated by the reservoir with outlets open during the entire flood with spillway crest as the maximum pool elevation and empty reservoirs as the initial condition.

(b) The degree of flexibility desired for most efficient use of reservoir storage where the outlet is gate-controlled.

(c) The stream flow that must be passed during the construction period.

(d) The time required to empty the reservoir.

(e) Discharge characteristics of outlets.

92. Outlet design flood.- From the standpoint of outlet design, volume of the flood and its duration are its two most important characteristics. The flood volume adopted has a probable frequency of recurrence of once in 100 years. It ranges in volume from 3.7 inches on 692 square miles to 9.6 inches on 17.3 square miles, which is the range of drainage area for the reservoirs under consideration. The flood hydrograph was constructed at each dam site from its distribution graph with a two-and-one-half-day occurrence of rainfall for which the distribution of intensity was triangular.

93. Selection of type of reservoir.- The decision as to the type of outlet (gate-controlled outlets or automatic, uncontrolled outlets) was based upon the time relation between flood run-off at the dam site and at centers of flood damage. For any site, where under reasonable conditions of rainfall occurrence, the flow from uncontrolled outlets during the automatic emptying of the reservoir might increase the natural peak discharge at the damage centers, it was considered necessary to provide outlet gates. The type of control selected for each reservoir is shown in Table No. 15. As would be expected, the retarding basins are those reservoirs located on tributaries remote from the centers of damage.

94. Determination of maximum outlet discharge for design flood.- For all reservoirs the maximum outlet discharge, considering for the gate-controlled reservoirs that all gates were open, was computed by trial and error to meet the requirement that spillway crest elevation should be the maximum pool elevation for the outlet design flood. This was done by routing the flood through the reservoir for various sizes of outlets, determining the maximum pool elevation for each trial, and

interpolating for the outlet size in terms of discharge that corresponded with the elevation of spillway crest. These values are shown on Table No. 15.

95. For the reservoirs with gate-controlled outlets the maximum outlet discharge should be greater than that described above by the degree of flexibility in operation from a practical standpoint. In general the nearer that a reservoir is located to a damage zone the greater are the possibilities of passing the early part of any flood through the outlets and conserving the storage capacity of the reservoir until its use would effect the greatest reduction of peak discharge at the damage center. From a study of dam site flood hydrographs routed through the natural valley storage of the Connecticut River to the main damage centers, the factors by which the maximum outlet discharge described above were multiplied to obtain the outlet design discharge were determined and are presented in Table No. 15. The range of variation of this factor is from 1.0 to 1.7.

96. <u>Provision for maximum flood discharge during construction</u>.- The design discharge based upon operating requirements for the individual reservoirs varies from 23 c.f.s. per square mile to 34 c.f.s. per square mile. It is believed that this is larger at all earth dam sites than the maximum flood discharge that should be conservatively provided for during construction, except at Newfane, North Hartland, Union Village, and Claremont, where provision was made to continue river channel flow during construction for all but the final few months in the summer season. At the masonry dam sites no additional provision is needed for the flood flows during construction.

97. <u>Time required to empty reservoir</u>.- The maximum time required to empty any of the reservoirs with the design discharge at spillway crest already described is approximately ten days. This was considered

to be ample, and therefore no additional provision for this factor was made.

98. Discharge characteristics of outlets.- The outlet area required to pass the design discharge was computed from the following formula:

$$A = \frac{Q\sqrt{1 + K_i + K_f}}{\sqrt{2gh}}$$

where A = Cross-sectional area of outlet in square feet.

Q = Outlet design discharge in c.f.s.

h = Gross head in feet. Elevation of spillway crest minus elevation of top of outlet for long conduits and minus elevation of center of outlet for short conduits.

K_i = Coefficient of intake losses.

= 0.10 for automatic outlets.

= 0.15 for gate-controlled outlets.

K_f = Coefficient of friction loss

$$= \frac{2gLn^2}{2.208\,R^{4/3}}$$

where L = Length of conduit in feet.

n = Coefficient of roughness as used in Manning's formula, taken here for smooth concrete as 0.13

The solution of the above equation is given in Table No. 15.

99. Number and size of gates.- The minimum requirement for number of gates was set at two in order to permit practical operation throughout a wide range of types of flood. The required gate area was taken as approximately 20% in excess of the recommended outlet area, with an even greater margin where dictated by excessive velocity. The

sizes and types of gates are shown in Table No. 15, and were selected to meet the best standards and greatest economics in design.

100. Plan of operation.- Reservoirs with uncontrolled outlets will operate automatically and are expected to fill on the average once in 100 years. The present designs contemplate outlets at the elevation of the stream bed and spillway crests with constant elevation. Although this gives fair efficiency of use of the reservoirs at medium floods, it is believed that detailed design studies will show the desirability of some alteration of this arrangement, in the direction of outlets at more than one elevation and possibly of spillways with staggered crest elevations below the presently considered spillway crest elevation which would be planned to discharge a small part of the outlet design flood. The overall effect upon cost of such tentative changes would be small and their adoption would depend upon detailed verification of the assumption of increased efficiency of use of storage capacity.

101. The reservoirs with gate-controlled outlets would be operated to obtain the greatest reduction of peak discharges at the damage centers below them. Continuous study of existing flood data and that from future occurrences will form the basis for detailed operating regulations. The reservoirs constructed for flood control alone will be held empty at all times when the stages in the river below them are less than the damaging stages, or at least to the heads on the outlets required to pass these flows. After a flood has receded below the damaging stages, the reservoirs will be emptied at a rate that will keep the modified discharges below them within channel banks. For the reservoirs in which conservation storage is included, that part of the storage reserved for flood control will be subject to the method of operation described above.

POOL ELEVATION FREQUENCIES

102. Frequency of recurrence of pool elevations.- A graph of pool elevation versus probable frequency of recurrence was determined for each reservoir. The basic data used in deriving them were the relations of pool elevations versus reservoir capacity, measured from the reservoir topography maps, and of flood volume versus probable frequency of recurrence, interpolated from similar relations for gaging stations with long periods of record. Individual relations of flood volume to reservoir capacity utilized were estimated for the reservoirs, which fall into two general classes: retarding basins with uncontrolled outlets and detention basins with gate-controlled outlets. The proportions of floods of various magnitudes that will be stored by each reservoir are shown on Plate No. 32. For the first class, the entire capacity of the reservoir below spillway crest is utilized during the flood with a 100-year volume frequency, according to the fundamental criterion applied in the determination of size of outlet, as described previously. For the second class of reservoirs, it was assumed that the entire flood at the dam site would be stored up to within one to one and one-half inches of reservoir capacity, and that for greater floods an increasingly larger proportion of the reservoir capacity would be utilized until, for the flood with a 100-year volume frequency, spillway crest would be reached. The graphs of pool elevation versus frequency were derived from the inter-connected chain of variables, frequency, flood volume, reservoir capacity utilized, and pool elevation. The graphs are shown on Plate No. 32.

FLOOD CONTROL

CONNECTICUT RIVER VALLEY

REPORT OF SURVEY

AND

COMPREHENSIVE PLAN

FLOOD LOSSES, AND

BENEFITS FROM PROTECTION

SECTION 2 OF THE APPENDIX

VOLUME 1

1. Introduction.- The Connecticut River Valley is often visited by floods, which have caused heavy losses. Flood losses will become increasingly severe because of the progressive urban and industrial development. The flood of November 1927 and the flood of March 1936, described in Section 1 of the Appendix, resulted in severe losses, which provide the basis for estimating average annual flood losses, and the economic justification of protective measures. Data on the 1927 flood losses have been taken from House Document No. 412, 74th Congress, 2nd Session. Data on the 1936 flood losses were assembled by a thorough investigation in collaboration with agencies of the various states. The watershed was divided into damage zones; losses were segregated into recurring and non-recurring losses; recurring losses were allocated to the damage zones; and the variation of loss with stages determined. From this stage loss relationship of recurring losses and results of the hydrological studies the average annual loss for a given degree of protection was determined.

2. Definition of direct and indirect losses.- Flood losses are grouped into two general classifications; namely, direct losses, and indirect losses. Direct losses are those resulting from physical damage to property or capital goods, and may be measured by the expenditures necessary to replace in kind. Indirect losses, though the result of direct damages, are not localized and are primarily concerned with the value of service and use, either lost or made necessary by reason of flood conditions.

3. Types of direct flood losses.- Direct flood losses were summarized under the types used in the basic project report, House Document No. 412, 74th Congress, 2nd Session, which are as follows:

Urban losses include losses of homes and places of habitation located in towns and cities, losses of sanitary and water supply facilities, damages to educational and religious institutions, parks and playgrounds, and miscellaneous municipal losses.

Rural losses include similar losses as indicated for urban areas but not located in towns and cities, and in additional land, crop and livestock losses.

Industrial losses cover all manufacturing light and power developments, telephone and telegraph facilities, fuel and petroleum products losses, etc.

Highway losses include all roads and pavements with appurtenant drainage structures, bridges and viaducts, and highway transportation maintenance and operating equipment.

Railroad losses include track, right of way, bridge and culvert losses, loading, storage and terminal facilities, stocks and supplies, and train equipment.

4. Classes of indirect losses.- Indirect losses may be divided into three general groups as follows:

a. Losses related to the five types of direct losses, described in the preceding paragraph, chiefly effects of direct damage because these losses result mainly from loss of use and service by damage and inundation, and the temporary and emergency services made necessary because of such conditions.

b. Intangible losses, which result largely from mental reactions originating from adverse conditions and apprehension of future floods.

c. Depreciation of property, which is the result of all disorganizing influences because of floods.

5. Direct losses of 1927.- This flood caused damages to property,

estimated at $15,526,000 for the entire watershed and resulted in the loss of 21 lives. It is to be noted that losses were particularly severe in Vermont where 70% of the damage occurred. In the upper watershed most of the damage was to highways and railroads with the White and Passumpsic Rivers in Vermont and the Ammonoosuc River in New Hampshire suffering a major portion of the losses. In the lower basin conditions were quite different from those in the other areas, for about 65% of the estimated damages were rural, urban, and industrial, of which 90% was along the main river. Railroad and highway damages, on the other hand, were mostly on the tributaries. Table 16 gives a summary of 1927 flood losses by states; Table 17 a summary of losses by river basins, each subdivided into the five classifications described in Paragraph 3. Plate No. 33 shows graphically the distribution of losses in the watershed.

COLLECTION OF 1936 FLOOD LOSS DATA

6. Preliminary investigations.- In order to obtain reliable and detailed data of the flood, all available personnel were assigned to duty in the flood area to assemble hydraulic data, flood-loss data, and other information of a pertinent nature. The main Connecticut River and most of the principal tributaries were visited by field investigators, either during the height of the flood, or as near thereto as was possible. Numerous informal interviews were had with Federal, state and local officials, civil organizations, and representatives of private interests. Wherever possible, statements were obtained as to flood losses sustained in each community. Where no estimates were available at that time, but were in the process of compilation, arrangements were made for procuring such information, when available. Where local estimates for various reasons were not available, representatives of the Department made their own estimates of the losses, with statements as to their ideas of the reliability of such estimates.

7. Investigations for this report.- The preliminary estimates were followed by an extensive field and office study which had three objectives:

a. To arrive at thorough estimates of the direct losses sustained in the flood of 1936 in those damage zones of the main river and on those tributaries located below reservoirs in the Flood Control Plan for the Connecticut River, inasmuch as only losses in these localities would be subject to any reduction by means of flood-control reservoirs.

b. To ascertain the relationship between flood losses and corresponding river stages. Knowledge of such relationship was necessary before the value of the alleviating effect of flood control works could be determined.

c. To collect data, which could be used as a basis for estimating indirect losses, which in the aggregate reach serious proportions although they are much more widespread and often of an intangible nature.

Several methods of approach to the problem were relied upon:

a. Contacts were maintained with those agencies which in the past had been instrumental in gathering flood-loss data; additional contacts were formed with similar organizations with the idea of instituting such further investigations as could be negotiated.

b. Correspondence with many interests in the damaged areas was relied upon to augment and clarify the many vague and brief references to loss which were the natural result of the hasty inspection during the height of the flood. By this means also many items of loss, not hitherto known, were discovered. In other instances, it was found that some of the early estimates were distorted. Those were revised in the light of the latest investigation.

c. Investigators were assigned to study, progressively and as thoroughly as time would permit, the physical conditions peculiar to each type of loss in each important damage center.

8. Cooperation of other agencies.- Whenever possible during the course of his investigation, the field investigator attempted to get from any available source a reliable estimate of the entire damage in that area, independent of any similar figure which might have been arrived at from other sources. Thus the data in the office files were given an approximate check and in many cases could be revised in the light of later developments. Since, however, organizations in many

localities expressly created for such purpose had made a thorough study of total direct losses, such information was used to a great extent and the efforts of the field investigators concentrated upon other lines of investigations. The exchange of information also gave assurance that no duplication of effort or waste motion would result. The fine degree of cooperation existing between representatives of this Department and several of the more active local agencies engaged in the compilation of similar data worked to the mutual advantage of both agencies.

9. Investigation of stage-loss relationship.- An important duty of the field investigator was the collection of data to disclose the relationship between flood stage and flood loss. For this purpose, the most important damages in each flood damage center were investigated. By means of a comprehensive questionnaire and with the aid of the owner or a representative, it was determined, for each property being investigated, at what point, below 1936 flood crest, damage began, and also, wherever possible, the increase in loss resulting from successive increases in flood stage until the crest was reached. As close an estimate as possible was also made of the additional loss which might result should a flood occur in excess of the previous maximum flood. In general, statements, of individual firms, home owners, farmers, utilities, etc., were accepted as being correct. Where there was considerable variance in estimates of loss, or owners were reluctant to make estimates, the representatives of this Department submitted their own idea as to the amount of loss.

10. General description of 1936 losses.- The unparallelled disaster and magnitude of damages resulting from ice and high water is manifested by the fact that in the large industrial and urban centers of the lower valley, stages were from six to eight feet higher than in

the 1927 flood and remained for about a week above the level of the 1927 flood which had been the greatest since about 1850. The flood was responsible for the loss of at least 11 lives by drowning and one by suicide. Nearly 10,000 homes were inundated, thousands were made homeless, industrial plants were damaged, traffic routes were interrupted, agricultural lands were destroyed by erosion and silting and much livestock was lost.

11. Amount and distribution of 1936 direct losses.- The total direct losses are estimated at $34,500,000. Tables which show the distribution of this total by States and other subdivisions may be briefly described as follows:

Table 18 - Direct losses by towns, State of Vermont;

Table 19 - Direct losses by towns, State of New Hampshire;

Table 20 - Direct losses by towns, State of Massachusetts;

Table 21 - Direct losses by towns, State of Connecticut;

Table 22 - Direct losses by States;

Table 23 - Direct losses by river basins.

Plate No. 34 shows graphically the distribution of 1936 direct flood losses.

12. Losses to agriculture in 1936.- Table 28 gives an indication of inundated areas and the extent to which agricultural lands were damaged by erosion and silting. This condition is quite serious in a district where all arable areas are fully developed.

13. Benefits of precautionary measures.- Losses would have been much larger had it not been for precautionary measures used and heeding of flood warnings. Complete figures are not available to show the value of damages prevented by flood warnings, but this may easily amount to 10% of the damages. Had not electric power failed, such savings would have been larger. The "Springfield Observer" stated that flood warnings resulted in a $1,500,000 reduction of flood losses in the Springfield,

Massachusetts area. City officials in Holyoke expressed the opinion that reductions of losses because of flood warnings were indeterminate, "but large". Investigations in Holyoke brought out the fact that much damage was avoided because it had been possible to move stocks, especially in paper mills, to upper floors.

14. Comparison of 1927 and 1936 flood losses.- The 1936 and 1927 flood losses were differently distributed between localities and groups. In 1927, the upper regions suffered more severely, while in 1936, the major damages were sustained in the highly developed and thickly settled lower valley. Damages to highways and railroads, although of approximately the same extent, were responsible for only 27% of the 1936 losses as against about 66% of the 1927 losses. In the upper reaches some of the highways, which had been damaged in the 1927 flood had been reconstructed at higher levels and of a better type. Some railroad branch lines in the upper reaches were no longer in existence at the time of the 1936 flood. Several industries and urban structures which had suffered heavy losses in 1927 never had been reconstructed. Old type bridges have been replaced with more modern types, stronger and with smaller channel encroachments. This may account for some of the decreases in damage in the States of Vermont and New Hampshire, but the main reason lies in the fact that discharges and stages were lower in 1936 than in 1927 in the territory above White River Junction, Vermont. The 1936 flood in the lower reaches so far exceeded all previous records, that in spite of moving many pieces of equipment, furniture and stock to levels above the 1927 flood, damages in the States of Massachusetts and Connecticut were over 10 times those caused by the 1927 flood. Plates Nos. 35 to 40, inclusive, show the distribution of direct losses graphically by States and classification and afford a comparison of 1927 and 1936 flood losses.

15. <u>Division of watershed into damage zones</u>.- The Connecticut River watershed was divided into 39 tributary and ten main stem damage zones by applying the following criteria:

<u>a</u>. Segregation of locations with high concentrations of damage.

<u>b</u>. A fairly constant relation between stage at an index point and flood damage in the entire zone.

<u>c</u>. A good relation between stage and discharge at one point within the zone. This necessitated at least one zone for each existing power pool on the main stem and occasionally an additional zone when a large tributary entered the pool.

<u>d</u>. The segregation of damage on tributaries in such manner that individual reservoir effects could be determined.

<u>e</u>. When not limited by the above considerations, zone limits were selected at township boundaries to facilitate field damage investigations and, in addition, zones were terminated where the main stem crossed the state boundary line.

These zones are given in Table 24 and their locations are shown on Plate No. 41.

16. <u>Modification of direct losses by flood control works</u>. - Direct losses which by their nature may be evaluated with a reasonable degree of accuracy were used as the basis for the determination of average annual losses. A careful analysis of these losses was made to determine that portion which is susceptible to modification by flood-control works. Losses occurring on streams above reservoir sites and storm damages on minor streams for which flood control cannot be econimically

justified were eliminated from summaries of recurring losses. Where there was positive knowledge that losses from some future flood would be materially different from those caused by the 1936 flood, estimates of such expected losses were supplemented. Thus, losses which were clearly non-recurring were eliminated. In this class fall losses to highways, bridges, buildings or equipment which were damaged by the 1936 flood, but since have been either abandoned or reconstructed in such a manner that future damages by floods may be expected to be less severe. Conversely, some damage figures were increased, as for instance in the case of factory buildings which were temporarily unoccupied at the time of the 1936 flood, but have since been reoccupied, so that a flood at the present time could be expected to effect damages over and above those caused by the 1936 flood because of the probability of damages to machinery and contents added since the 1936 or demonstration flood. Deductions for non-recurring losses, however, were not carried to the extreme; for it is important not to lose sight of the possibility of new development and therefore additional potential damages in the future. The recurring losses shown in Table 25 give consideration to these modifying influences.

17. Stage-loss relationship.- Stage-loss relationship was primarily based upon the 1936 flood experience, but 1927 data were given consideration, wherever they were known in sufficient detail to permit their allocation to damage zones. Individual stage-loss questionnaires, executed by field investigators, were segregated by villages, towns, and damage zones, and the losses classified as described in Paragraph 3. Losses at either critical or typical stages were recorded for the property being inspected. In general, the method of straight-line interpolation for each foot of stage intervening between adjacent stage-loss estimate was resorted to, unless the knowledge gained by the field

investigator indicated that other procedure was preferable. In this manner a summary was prepared, for each individual loss investigated, showing foot by foot the estimated loss for each stage from the beginning of damage to the crest of flood, and, in addition, the estimated damage which might result in case of some future flood of higher stage. The total loss for which stage relationship was determined for each type, in each town, was then compared with the over all total of that type of recurring loss in that town and the ratio of the investigated total to the over all total determined. Since it was intended that the losses investigated in any flood damage center be those which could be considered either typical or those which for other reasons influenced most strongly the shape of the stage-loss curve, it was considered to be permissible to multiply the sum of investigated losses at each foot of stage by the above-mentioned ratio. The stage-loss curve was thus expanded in proportion, and made to pass through the point for total recurring damage sustained for each flood damage center and type. Summaries of recurring losses for successive stages for towns were then tabulated by damage zones and finally a stage-loss relationship curve prepared for the estimated total of each type of loss for each damage zone. As a final step, component curves representing the five types of losses were drawn, the curves added, and a total curve for all losses for the damage zone in question arrived at. Curves for damage zones for the 20-reservoir plan, arrived at as described above, are shown on Plate 42.

18. Indirect losses.- General.- During the period of flood devastation and subsequent reconstruction the normal routine of all activities in the valley was severely upset and the influence of this confusion reached far beyond the immediate flood zone. Crippled transportation facilities, discontinuity of utility services, curtailment of production,

suspension of normal commodity exchange, and emergency measures to safeguard life, health, and property existed in varying degrees. Mental reactions originating from adverse conditions and apprehension of future floods influence detrimentally, and will continue to do so in diminishing degrees for a long time to come, the recovery of pre-flood activity level and normal growth. The general upheaval and protracted recovery of adtivities are responsible for a large indirect loss of which only a portion can be measured in monetary values.

19. Indirect related losses.- Description.- Indirect losses that are susceptible to partial evaluation are those related to direct losses. They result from conditions arising from loss of use or service of material things either destroyed or damaged. The more important losses resulting from the above mentioned condition are enumerated as follows:

a. Loss of normal business and production to establishments directly damaged, their suppliers and customers.

b. Loss of wages to employees in completely or partially shut-down industries and places of business.

c. Loss of good will and permanent loss of business.

d. Loss of income by reason of low rent or curtailed dividend payments.

e. Extra cost of carrying on business under adverse flood conditions.

f. Cost of regaining temporarily lost business.

g. Extra cost involved in replacing damaged stock and equipment.

h. Cost of traffic detouring and delays.

i. Loss caused by interruption of utility services.

j. Expenditures to alleviate distress conditions, prevent sickness and epidemics; for sanitation, policing, and ferrying.

k. Cost of capital needed to replace direct losses.

20. Interruption of normal business.- One of the most serious causes for indirect losses was the interruption of normal trade channels and isolation of many communities for days and in some instances weeks. This sudden cessation of normal activities affected the entire communities and not only portions actually flooded. Losses of this type are not easily evaluated, although in the aggregate they were substantial, and were felt in many parts of the country. To cite one instance: work on the Passamaquoddy and Fort Peck projects was delayed to some extent because cables and machinery parts originating in the flooded area could not be delivered according to schedule. In this class of loss falls the inconvenience of interruption of power service with its possible danger of accidents, delayed communications in case of sickness and injury upon which no money value can be placed. Telegraph companies were able to partially maintain service only by use of emergency power reverting to the former and slower Morse system when operation of automatic telegraph machines had to stop because city power service failed. A very costly effect of interruption of power service was the stopping of elevators which prevented the movement of valuable stock and merchandise to higher levels and thereby materially increased direct losses. Production schedules in many factories located in various parts of the country were upset, either because manufactured goods could not be delivered, or needed parts were not received at the proper time.

21. Investigation of indirect related losses.- Many who were interviewed in the effort to ascertain the extent of indirect losses were willing and able to estimate the extent of their indirect losses. In general, these estimates were accepted as being correct under the assump-

tion that the informant's knowledge of his own property was more reliable than that of the investigator. It should be stated, however, that there were instances where much reluctance was shown to fully disclose the seriousness of losses because of fear that publication of heavy losses would have an adverse effect upon credit standing and incidentally might cause unfair competition. In order to get a better idea about the extent of the effect of interruption of normal activities because of the flood, a number of firms in various parts of the country were contacted. These firms normally furnish goods to industries within the flooded areas. Only a few of the firms thus contacted, attempted to estimate indirect losses in terms of money loss. The majority simply stated that they did suffer losses, which they were unable to evaluate, but did describe as "serious" and were sustained because of the following reasons:

- a. Loss of normal share of business.
- b. Not being able to make deliveries or get products originating in the flooded area.
- c. By having to change their production schedule, taking down and setting up special machinery.
- d. By having to lay off some of their employees.
- e. By having to replace goods damaged in transit, which in some instances necessitated use of premium time.
- f. Shipment had to be made over longer routes by trucks instead of over normal routes.
- g. Collection of moneys due from firms in the flooded area was slow.

In practically every instance it was stated that although these indirect losses were sufficiently large to be felt, it was impossible to estimate their money value. Some few did supply estimates and from these and information gained in informal interviews it is believed that indirect losses of the above type amount to at least 10% and may well reach as high as 30% of the direct losses.

(Report continued on following page)

22. Examples of indirect losses.- It may be well to cite in more detail one case, possibly an extreme one, where more or less complete estimates were received. It concerns an industry with plants on the Connecticut River in Massachusetts. The plants were flooded and this industry reported a direct loss of $98,000. They estimated their loss of sales at $50,000 and possible wage loss to employees at $25,000 or a total estimated loss of $173,000.

Four of their more important suppliers were contacted and made replies to the questionnaire sent to them. One of the firms (located in Boston, Mass.) reported that their indirect loss, because of the shut-down of the plants of above and other customers in the flooded area, is estimated at from $150,000 to $175,000. In addition to this, all others reported serious interference with their business which they could not express in money value.

Two of these reported that bills amounting to $27,000, now long overdue, cannot be collected, because the industry located in the watershed is now in the hands of a receiver, which condition was at least partially caused by the severe flood losses.

23. Urban and industrial indirect losses.- Indirect losses associated with industrial and commercial enterprises were studied in the most detail. Limited time, though, permitted contacts with only the larger establishments. Some few of these were of the opinion that no losses of the indirect type other than delays were sustained. Others were unable to determine whether any indirect losses were involved or of the opinion that such losses were later offset by increased activities. Many indicated that in addition to the direct losses which they reported, substantial indirect losses were sustained but because no detailed cost records were kept they were unable to evaluate such losses in a more definite manner than to state they were noticeable or severe.

Finally a fourth group gave estimates of both direct and some of the indirect losses enumerated in paragraph 19. Some 1215 cases representing a direct loss (industrial and urban commercial) of $12,765,000 or 56% of the total of this classification were analyzed. Of these only 41 say definitely that there was no indirect loss; 646 make no comment on indirect losses, and the remainder, having direct losses of $3,919,230, estimate that indirect losses, because of loss of business, loss of good will, and loss of wages to their employees, amount to $4,468,405, indicating a ratio of 1.14 between indirect and direct losses.

It is believed that the same ratio of direct to indirect losses will apply to the remainder of industrial and urban losses which were not investigated.

24. Highway indirect losses.- Traffic counts of all classes of vehicles for a 24-hour average day in March 1936 were available for a majority of traffic routes in the State of Massachusetts and formed the basis of an estimate of the economic loss caused to vehicular traffic in the Connecticut Valley during the 1936 March flood, when all except the Gill-Erving, French King Bridge, crossing the Connecticut River were closed to vehicular traffic.

The cost of operation per vehicular mile is taken at $0.0544 and the cost of delay is assumed at $1.00 per vehicle hour, 8 hours a day for 4 days, the average time bridges were closed for traffic.

With these assumptions, the economic loss because of detouring is estimated at $44,280 and the economic loss because of delays at $2,313,568, or a total loss of $2,357,848, within the State of Massachusetts.

Compared with the total direct highway losses in the State of

Massachusetts, which are estimated at $4,774,000, that portion of the indirect losses which was evaluated above, amounts to almost 50% of the direct highway losses. In addition to these losses, which primarily affect operators of motor vehicles, are the losses to businesses along established traffic routes, which were inoperative for varying periods of time, and the costs of maintaining and restoring inferior roads pressed into service to accomodate the heavy traffic, for which these roads were inadequate.

Traffic counts are not available for the other states affected by the flood, but it is not unreasonable to assume that a similar ratio between direct and indirect losses would be found in other localities.

25. Railway indirect losses.- Some indication of the extent of indirect losses sustained by railroads may be gained from a comparison of gross and net earnings of the four more important railroads serving the Connecticut River Basin. From statistics for these 4 railroads, i.e. N.Y.N.H. & H., B. & M., M.C., and C.V., contained in the reports of the Bureau of Railway Economics, it is found that gross earnings in 1936 are slightly above those of 1935. The net earnings, however, show a drop of over $3,400,000, for the months of March and April 1936 under 1935 and practically the same drop for the period March to November, showing the drop for the period March-April to be an extraordinary charge produced by other than normal operating conditions. The flood in the Connecticut River Basin and other parts of New England, of March 1936, is at least partially responsible for this drop in net earnings.

In the financial statement recently published by the B. & M. R.R., it is stated that: "Due to flood damage of upwards of $2,000,000, suffered last spring, all of which was charged into current accounts during the year, the B. & M. R.R., in 1936, showed a deficit after fixed charges of $1,654,182. If it had not been for the flood, (it was pointed out) the roads would have had a net income after charges of well above $400,000." In addition, the road's statement said, "There is no way of determining the amount of revenue lost during the flood when both passenger and freight service was suspended during the flood and repair periods."

Gross earnings of the four major New England R.R. Companies during the months of February, March, and April 1936 were larger by $772,000, $57,000, and $752,000 respectively, than gross earnings for the corresponding months of 1935. On the other hand, operating expenses for the same months of 1936, exceeded those of the like 1935 periods by $1,118,000, $2,117,000, and $1,482,000, respectively, indicating a loss of revenue of approximately $700,000 and an increase in operating expense of approximately $1,000,000 during the month of March 1936. Since the increase in operating expenses is primarily due to extraordinary outlays because of flood and the drop in operating revenues also is primarily due to loss of revenues because of the flood, a comparison of these two figures will be somewhat indicative of the ratio between direct and indirect losses. Besides these losses to the railroads themselves, the affects of interrupted or irregular service were widespread and caused innumerable other losses to industry, commerce and individuals.

The above figures indicate that indirect losses may be taken to be about 70% of direct losses.

26. Agricultural indirect losses.- Reduction of crop yield because of top soil erosion and heavy deposits effecting approximately 4600 acres of cultivated land is estimated at 20% of the total annual yield over a period of 5 years, the length of time estimated

for such lands to reach their pre-flood fertility. An estimate of $30 per acre as an average annual yield would reflect on indirect loss for the five-year period of $138,000. Additional indirect losses resulting from the loss of sales of products, particularly dairy products, extra cost of replacing special breeds of livestock will conservatively place the indirect loss **equal to at least 10% of the direct rural** loss.

27. Average ratio of related indirect to direct losses.- Upon the basis of the ratios developed in the preceding paragraphs, namely - 114% urban and industrial, 50% highway, 70 % railroad, and 10% rural it is estimated that the indirect losses below the reservoir sites considered amount to $30,410,000 or 94-1/2% of the total direct recurring losses of $32,257,000 below these sites. This estimate is believed to be most conservative as it is based on only those types of losses within the flooded areas, which are susceptable to evaluation in money equivalent, but exclude those sustained by establishments or individuals of unknown identity outside the inundated areas, who suffered only indirect losses. Table 27 summarizes direct and indirect losses by damage zones.

28. Indirect intangible losses.- Intangible losses of great economic importance result from the following conditions:

a. Possibility of loss of life or impairment of health.

b. Mental distress caused by losses and apprehension of repetition of flood.

c. Inability to rent or sell property because of the possibility of a recurrence of another disastrous flood.

d. Stopping of normal industrial expansion or additional development.

e. Exodus of industries from flooded areas.

f. Effect upon social security of inhabitants.

g. Property is not used to its highest utility although there is a potential demand for such development.

29. Cost of disease prevention.- Barring minor ailments caused by exposure no extensive impairments of public health were sustained. Substantial extraordinary expenditures by local health departments were necessary to prevent outbreak of epidemics. State public health departments for the States of Connecticut and Massachusetts alone report extraordinary expenditures for preventive measures of about $200,000. Problems of severe exposure, contaminated water and food supplies, and debris flooded houses were successfully solved by the emergency measures pressed into service. Only through precuationary measures, such as chlorination of water supplies, discarding and replacing submerged food, disinfecting of household furnishings and houses, and immunizing by inoculation thousands of persons against typhoid and other contagious diseases, was it possible to prevent widespread impairment of health.

30. Depreciation of property values.- The hitherto unbelievable precedence for great floods on the Connecticut River as established in 1936 was severely impressed upon the minds of the inhabitants in the valley. From these impressions of flood disaster, such as loss of life and property, discomfort and mental anguish, indirect losses to business, and other intangible losses, and their possible recurrence, there originate permanent disorganizing influences resulting in losses of values in the inundated areas. These losses result from a partial or general exodus from the severely flooded areas, of industries and other activites, of capital, and finally of inhabitants. In the less severely affected areas these effects begin or further a process of degradation by such causes as curtailment of industrial expansion or development, inability to secure credit facilities for construction or repair, flow of capital from the area, inability to rent or sell property,

reduced taxability, and stopping of normal growth. Finally, the partial abandonment or degradation of the flooded area results in depreciation of property values. Prevention of recurrence of flood losses will check further depreciation of property values and disintegration of existing business and living conditions in the flooded area and ultimately result, not only in restoring property values to pre-flood levels, but open the way to further appreciation of values because of change in the utility of such tracts within the flooded area as at the present time have not reached their best potential development.

31. Evidence of depreciation.- A considerable amount of correspondence on this subject is on file. The following lines summarize the more important comments made by persons contacted for the purpose of formulating a basis of an estimate of the extent of such property depreciation. Bankers, real estate men, and owners of rental property are unanimous in their statements, that many properties within the flooded areas cannot be sold or rented because of the fear of a repetition of the last disastrous flood. Industrial expansion is at a standstill; many instances are known where contemplated expansions have been deferred until adequate flood protection will be provided. Many industries have stated, that one more flood like the one of 1936 will bring about complete cessation of their activities and abandonment of their plants. The Federal Housing Administration and private banks consistently refuse loans to properties within the flooded area. The Home Owners Loan Corporation, as well as other loaning agencies reports that a more than normal share of mortgaged properties located in the flooded areas is in default. The Springfield, Massachusetts Chamber of Commerce expressed very clearly the apprehension which exists in many industrial communities about the continuance of industrial

activities and their comments are quoted in full:

"It is impossible to estimate with anything approaching accuracy the amount which this item might reach. It is doubted if expansion actually has been greatly retarded in anticipation of damaging floods, but great apprehension still exists and there has been some moderate shifting of operations from the flooded area to other locations. If a similar experience should occur at an early date it is believed that it would result in the removal of industrial activities which might account for a volume of production running from $25,000,000 to $50,000,000 a year. Several manufacturers have indicated that they are willing to take one more chance - but only one."

Real estate men, assessors, bankers, and individuals were interviewed to get their ideas to what extent property depreciation has taken place in their particular communities and were almost unanimous in stating that property values in the flooded areas dropped about 20% to 25%. This computation of depreciation does not include the capitalized value of future average annual direct and indirect flood losses described in paragraphs 45 to 48, inclusive, of the report.

32. Estimate of depreciation in values.- For the Connecticut River Watershed as a whole, property value depreciation because of floods may conservatively be estimated at approximately $75,000,000, or more than twice the direct flood losses caused by the 1936 flood below proposed reservoir sites. If complete flood protection were provided, losses because of depreciation of property values would be eliminated within a short time, probably before construction of protective works is completed. Without flood control these values would partially recover, provided another disastrous flood did not occur for a number of years. Making allowance for this condition and also giving consideration to the fact that some recovery of values would result from the efforts of chambers of commerce, real estate operators, and others vitally interested, there remains nevertheless a substantial loss, estimated at 80%, which can be eliminated only by complete protection from future

disastrous floods. Upon the assumption that real estate should bring a minimum return of 6%, the estimated annual loss which can be avoided by complete protection from future disastrous floods then would be not less than 6% of 80% of $74,857,000 or over $3,593,000.

33. Summary of depreciation.- Table 26 affords a comparison of 1936 direct flood losses and the capital loss because of property depreciation in some of the more important towns in Connecticut and Massachusetts. Tables 29 and 30 show the estimated depreciation of property values, summarized by damage zones (below proposed reservoir sites) and States.

34. Conclusion.- The very nature of indirect losses defeats their evaluation by a process of mathematical summation. Such losses are too far distributed. They are of such nature that even were it possible to contact the majority of losers it would be difficult to get complete figures because of the reluctance shown by many informants to estimate such losses in terms of money values. However, it is believed that such estimates as have been given, may be taken as indicating that indirect losses, exclusive of those reflected in a property depreciation, are substantial. Summarizing the influences of all elements having a bearing upon the subject and giving careful consideration to all available data, observation made during inspection trips and talks with many inhabitants of the flooded areas, the conclusion is reached that indirect losses, exclusive of property depreciation, at least equal direct flood losses. In determining the justification of flood control works, however, the determinable ratio of 94.5% of indirect to direct losses is used. Table 27 summarizes by damage zones the direct and indirect losses and depreciation of property values.

DETERMINATION OF FLOOD CONTROL BENEFITS

35. Basis of economic benefits.- The benefits from flood control are derived from the reduction of direct and indirect losses, and from the restoration of the depreciation of property values caused by a flood and sustained by the apprehension of the recurrence of damage from floods. Definite relations of direct flood loss to stage were developed for all damage zones of the Connecticut River Valley as heretofore described. These relations, in combination with flood histories and hydrological data, form the basis for determining average annual flood losses. The flood reducing effects of the group of reservoirs of the Comprehensive Plan, and of individual reservoirs, evaluated as described heretofore in Section 1 of the Appendix, were combined with the above data to obtain average annual flood losses modified for the reservoirs in operation. The successive steps in this analysis and the results obtained are described below. The determination of the average annual benefit from the dikes is explained in Section 5 of the Appendix.

36. Determination of stage-discharge relations.- Stream gaging stations of the United States Geological Survey were available for use as index points in five main stem and twelve tributary damage zones. The rating curves for these stations, Bellows Falls and Vernon power dams, Springfield, and Hartford are shown on Plates Nos. 2 and 3. For the remainder of the tributary damage zones it was necessary to construct rating curves at the index points. This was accomplished by use of the following data: low water, November 1927 high water, and the March 1936 high water profiles; and the discharges accompanying them from estimates of flow over dams, corrected for run-off from intervening drainage areas when the index point was not directly at a dam, and from rainfall and melting snow cover by the use of distribution graphs determined

from watershed topographical characteristics as described in Section 1 of the Appendix.

37. Determination of peak discharge-frequency relations.- The method of determining peak discharge-frequency relations at United States Geological Survey gaging stations having 12 or more years of record is described in Section 1 of the Appendix. The peak discharge-frequency relations at the ungaged index stations, and at those gaging stations with short periods of record, were obtained as follows:

a. From the flood volume - drainage area - frequency relation for the Connecticut River Watershed shown on Plate No. 11, a graph of flood volume-frequency was extrapolated for the drainage area of the index station.

b. The frequencies of the November 1927 and March 1936 floods were obtained by interpolation of the computed frequencies for those floods at nearby gaging stations.

c. The estimated peak discharges for the two floods were divided by the flood volumes of the same frequency and the average of these ratios applied to the flood volume - frequency relation to obtain the peak discharge - frequency relation. The constancy of the relation between peak discharge and flood volume is discussed in Section 1 of the Appendix.

38. Determination of average annual flood losses.- Direct flood loss was related to frequency through the continuous chain of interrelated variables developed above, namely: direct flood loss to stage, stage to peak discharge, and peak discharge to frequency. The resultant relation for each zone was plotted on linear coordinates with direct flood loss as the ordinate and frequency in terms of probable per cent chance of recurrence in one year as the abscissa. The average annual

direct flood loss was obtained by computing the mean ordinate of this curve for the 100 per cent chance abscissa. The direct flood loss-frequency graphs for all damage zones below the reservoirs in the Comprehensive Plan are shown on Plates Nos. 43 to 48, inclusive. The average annual indirect losses were computed as 0.945 times the direct losses on the basis of the relation between the direct and indirect losses sustained during the March 1936 flood. Average annual losses from depreciation of property values were computed as 4.8 per cent of the estimated depreciation at the present time. This was based upon a conservatively estimated net return on property of 6 per cent per year, and an estimated average depreciation, over a long period of time, of 80 per cent of the depreciation caused by the flood of 1936. Average annual flood losses for all damage zones in the Connecticut Basin are given in Table 31.

39. <u>Determination of average annual benefits from the reservoirs of the Comprehensive Plan.</u>- Average annual modified direct flood losses, remaining after peak discharges were reduced by reservoir storage, as described in Section 1 of the Appendix, were computed by placing in the above mentioned chain of interrelated variables the relation of modified peak discharges to frequency which produced the resultant graph of modified direct flood loss versus frequency. The mean ordinate of this curve for the 100 per cent chance period produced the average annual modified direct flood loss. The direct benefit from average annual reduction of direct flood loss by the reservoired storage was measured as the differential between the natural and modified mean ordinates. The average annual indirect benefits were obtained by applying a factor of 0.945 to the average annual direct benefits. The average annual benefit from restoration of property values was taken as the

average annual loss from depreciation of property values, not including those areas where supplemental protection by dikes is proposed. The reductions of all average annual losses and the reductions of recurring direct losses for the November 1927 and March 1936 floods by the reservoirs of the Comprehensive Plan are given in Table 31.

(Report continued on following page.)

Determination of Average Annual Benefits to Individual Reservoirs

40. Basic method of determining direct benefits.- The main river average annual direct benefits for each reservoir were obtained by summating its benefits in each damage zone as determined from the formula, $B = C_W U L R$, in which:

B = Average annual direct benefit in dollars.

C_W = Per cent reduction of peak discharge provided the entire flood is stored.

U = Average annual direct benefit for a one per cent reduction of peak discharge.

L = Ratio of reservoir capacity in inches to flood volume in inches at index station of damage zone.

R = Portion of zone, expressed as a ratio, that is affected by a given reservoir. ($R = 1.0$ unless some part of the zone is not below the reservoir.)

The computation of the C_W values is described in Section 1 of the Appendix.

41. Determination of U and L.- The following analysis of average annual benefits from reduction of direct flood loss was made for each damage zone to determine the U values. By the method described in Paragraph 38, graphs of modified direct flood loss-frequency were computed for reductions of the natural peak discharge of 5, 10, 20, 30, and 40 per cent. Since the flood-controlling effect of any reservoir becomes less and less as the flood volumes of rare frequency exceed the reservoir capacity by increasing margins, the per cent reduction of peak discharge will decrease accordingly. In order to evaluate this variable the areas under these graphs were divided by vertical lines of constant frequency into several components varying in number from two to five depending upon the magnitude of the total average annual direct flood

less for the zone. The average annual direct benefit within each component of frequency range was computed for each of the parameters of per cent reduction of peak discharge. The results were plotted with per cent reduction of peak discharge as ordinates and average annual direct benefit as abscissae. The slope of this graph for the range of per cent reduction of peak discharge is the U value for that one reservoir for one frequency range. These graphs for the frequency ranges of all damage zones are shown on Plates No. 43 to 48 inclusive. It can be seen that the benefit per unit of reduction of peak discharge varies inversely with the degree of reduction of peak discharge. The benefit to any reservoir will depend, therefore, to some extent upon the assumed prior reduction of the peak discharge at the index station. L was obtained as an average value for each component of the frequency range by the following formula, which is an adaptation of the prismoidal formula:

$$L_{avg} = 1/4\left[\frac{S}{V_B} + 2\frac{S}{V_M} + \frac{S}{V_E}\right]$$

in which

V_B = Natural flood volume in inches for lowest frequency of the component of frequency range.

V_M = Natural flood volume in inches for mean frequency of the component.

V_E = Natural flood volume in inches for highest frequency of the component.

S = Capacity of reservoir in inches.

When any of the V's is less than S, the corresponding $\frac{S}{V}$ term is kept at unity, its maximum value.

42. Direct benefits in tributary damage zones were computed similarly with the exception that the individual reservoir benefits were read directly from the graphs of benefit-per cent reduction of peak discharge, making the U term unnecessary. When several reservoirs are located above a damage zone, the analysis is expedited by use of the U term. In Table 32 are given typical computations of the direct benefit from an individual reservoir and in Table 33 are summarized the direct benefits from each of the 20 reservoirs in the Comprehensive Plan and their alternates based on two premises: as the first reservoir in a system, and as one of a system with no order of preference.

43. Determination of other annual benefits.- Average annual indirect benefits from individual reservoirs were obtained by multiplying their direct benefits by 0.945. The average annual benefit of a system of reservoirs in restoring property values in each damage zone was allocated to individual reservoirs according to their proportionate flood reducing effects. The summation for each reservoir of component benefits in all damage zones below it produced its benefit in the restoration of property values. They are given in Table 33.

44. Relation of average annual individual reservoir benefits to reservoir capacity.- The factor, L, used in determining individual reservoir benefits is a function of reservoir capacity and flood volume. The capacity of each reservoir was varied from 4 inches to 9 inches by increments of one inch and the entire computation of its individual benefits, described in the preceding paragraphs, was repeated for each assumed capacity. A graph of annual benefit-reservoir capacity was prepared which formed, in combination with a graph of annual cost-capacity, the basis for determining the most economical reservoir capacity. It is

the capacity for which an incremental change will produce increments of benefits and cost that bear the same relation to each other as the total annual benefit for the best system of reservoirs bears to the total annual cost of the system.

FLOOD CONTROL

CONNECTICUT RIVER VALLEY

REPORT OF SURVEY

AND

COMPREHENSIVE PLAN

CONSERVATION FOR

POWER AND RECREATION

SECTION 3 OF THE APPENDIX

(VOLUME 1)

SECTION 3

CONSERVATION - POWER AND RECREATION

1. Scope.- In this section are presented a summary of the existing and prospective future power and storage developments and production of electric power in the Connecticut River Basin; the detailed analyses at sites of flood control reservoirs of power development and of added conservation storage for increasing low-water flows for the benefit of power plants below; and the recreational and sanitary values of added conservation storage. Reference is made to the main report for a general description of existing hydroelectric developments, production of electric power, existing and prospective future power plants, storage reservoirs for power storage alone and conservation storage developed with flood control projects. Much of the basic data herewith presented were obtained from House Document No. 412, 74th Congress, 2d Session, the Document No. 308 Report on the Connecticut River. The following information gives the results of recent additional studies and investigations that have been made on the flood control projects now under consideration.

POWER

2. Existing hydroelectric developments.- In the following table are given the location, head and capacity of each of the existing sixty-three hydroelectric plants within the Connecticut River Basin.

(Table on following page)

River	Location	Gross head, foot	Installed capacity, kilowatts
Connecticut	Canaan, Vt.	37.7	1,100
"	Lyman Falls, Vt.	31.6	1,000
"	Lower Fifteen Mile Falls, N. H.	176.0	140,000
"	McIndoes Falls, Vt.	32.4	10,000
"	Wilder, Vt.	37.0	3,120
"	Bellows Falls, Vt.	63.0	45,000
"	Vernon, Vt.	36.0	28,000
"	Turners Falls No. 2, Mass.	67.4	52,000*
"	Turners Falls No. 1, Mass.	**60.0	5,000*
"	Holyoke No. 1, Mass.	**24.0	7,080*
"	Holyoke No. 2, Mass.	**20.0	2,900
"	Holyoke (municipal), Mass.	**12.0	1,056
"	Windsor Locks, Conn.	34.4	170*
Ammonoosuc	Bethlehem, N. H.	46.3	300
"	Lisbon, N. H.	16.3	300
Ashuelot	Marlboro, N. H.	269.0	1,600
"	Swanzer No. 1, N. H.	16.0	120
"	Swanzer No. 2, N. H.	18.0	120
"	Troy, N. H.	16.4	150
Black	Cavendish, Vt.	120.9	1,500
"	Perkinsville, Vt.	22.3	368
Chicopee	Indian Orchard, Mass.	36.3	6,100
"	Bircham Bend, Mass.	16.0	750
"	Chicopee, Mass.	36.2	2,100
"	Blanchardville, Mass.	16.0	1,125
Deerfield	Searsburg, Vt.	230.0	4,700
"	Whitingham (Harriman), Vt.	390.0	45,000
"	Rowe (Sherman), Mass.	80.0	6,000
"	Florida No. 5, Mass.	240.0	15,000
"	Shelburne Falls No. 4, Mass.	64.0	6,000
"	Shelburne Falls No. 3, Mass.	66.0	6,000
"	Gardners Falls (Shelburne), Mass.	40.0	4,000
"	Shelburne Falls, No. 2, Mass.	60.0	7,000
Farmington	Tariffville, Conn.	32.8	1,800
"	Robertsville, Conn.	55.6	500
Israel	Lancaster, N. H.	25.0	128
Mascoma	Lebanon No. 1, N. H.	18.7	150
"	Lebanon No. 2, N. H.	16.2	140
"	Lebanon No. 4, N. H.	71.7	1,050
Millers	Winchendon No. 1, Mass.	20.5	350
"	Winchendon No. 3, Mass.	17.2	200
"	Wendell, Mass.	22.2	1,120
"	Farley, Mass.	18.0	360
Mill Brook	Windsor, Vt.	40.0	300
Passumpsic	Passumpsic No. 4, Vt.	24.1	700
"	St. Johnsbury No. 0, Vt.	17.2	250
"	St. Johnsbury No. 1-1/2, Vt.	19.1	350
"	St. Johnsbury No. 2, Vt.	9.6	150
"	St. Johnsbury No. 3, Vt.	17.0	875
"	West Danville, Vt.	171.3	1,000
"	Lyndonville, Vt.	15.3	60
"	" "	61.1	600
	Carried forward		414,742

River	Location	Gross head, feet	Installed capacity, kilowatts
	Brought forward		414,742
Salmon	Leesville, Conn.	23.0	390
Stevens	Barnet, Vt.	86.1	200
Sugar	Claremont, N. H.	24.0	250
"	Sunapee, N. H.	71.0	560
"	" "	58.0	470
Waits	Bradford, Vt.	73.4	360
Wells	Boltonville, Vt.	66.9	470
West	Dummerston, Vt.	21.1	620
Westfield	Cobble Mountain, Mass.	430.0	23,000
"	Westfield, Mass.	11.5	125
White	Royalton, Vt.	13.1	560
	Total		441,747

* Large volume of water sold to industries.
** Approximate.

This table shows that most of the smaller hydroelectric plants are located on uncontrolled tributaries of the Connecticut River. Many industrial plants, not included in this table, operated by water power are also located on tributaries.

3. Electric power development in "Zone".- The development of electric power in the Connecticut River Basin must be considered as an integral part of power development in the zone comprising all the New England States except Maine. This latter state interchanges but little power with other states, and since the Public Utility Act of 1935 became effective there has been little interchange of power between Connecticut and other states. The remaining states are a closely-knit unit as to power production and distribution, and the primary market for Connecticut River power is within the State of Massachusetts. The greater part of the electric power used in New Hampshire and Vermont

is furnished from smaller electric systems, with their plants located on other rivers and tributaries of the Connecticut River.

4. On the Connecticut River, north of Massachusetts, are four of the principal hydroelectric plants controlled by the New England Power System. These plants have a combined capacity of about 223,000 kilowatts. Owing to the limited amount of storage available, and the wide variation in the flow of the Connecticut River, these plants can be operated successfully only by being interconnected with other electric-power systems having generating facilities for developing large amounts of prime power when water is not available in the river, and absorbing large amounts of prime power when there is sufficient water to operate the hydro plants up to their full capacity. The same condition applies at the Cabot Station of the Turners Falls Power Company, having a capacity of about 57,000 kilowatts. The transmission lines of this company are interconnected with the lines of other power companies in the western part of Massachusetts. Corporate consolidations and control, together with intercompany agreements and other sources of power, for the purpose of exchanging large blocks of power on demand, and maintaining, as far as possible, continuous service, have brought together nearly all of the companies operating in the zone. The interconnection of the hydroelectric power plants located on the Connecticut River with the steam plants of the Edison Electric Illuminating Company in Boston, and other centers similar to Lawrence, Worcester, Springfield, and Providence, makes possible the utilization of practically all water flowing in the Connecticut River up to the installed capacity of the hydraulic turbines in the existing power stations. The installed capacity of hydroelectric and steam power plants in the zone producing electric

power for sale increased from about 106,000 kilowatts in 1899 to 1,035,000 in 1919, and to a maximum of 2,686,000 kilowatts in 1933. At the end of the year 1936 the total generating capacity was approximately 2,529,000 kilowatts, of which 616,000 kilowatts was hydro and 1,913,000 kilowatts was fuel. Many of the smaller steam plants are old and are used only as stand-by or on peaks of the load.

5. Production of electric power in "Zone".- The charts on Plate No. 50 indicate graphically the installed capacities of the electric power plants, and the yearly production of electric power by public utility plants in New England, and in all the New England States, exclusive of Maine, since 1924. It will be seen from Plate No. 50 the combined capacity of the hydro and steam plants is less than in 1932, and therefore with a continued increase in the demands for electric power, it will be only a question of time before additional generating capacity will become necessary. It seems reasonable then to expect that consideration will be given by power companies to the development of storage reservoirs on the Connecticut River, where their principal hydroelectric plants are located. These hydroelectric plants have a large reserve generating capacity during the greater part of the year. Any increase in the lower flows of the Connecticut River created by storage reservoirs may, therefore, be utilized without added equipment or expense of operation. Each cubic foot per second increase in the normal flow in the Connecticut River above the existing Fifteen Mile Falls Plant will produce approximately 25 kilowatts of electric energy in the five principal power stations above Greenfield, Massachusetts. The combined operating head at the five plants is about 364 feet.

6. Existing storage reservoirs.- As shown in the main report,

numerous storage reservoirs throughout the basin have been constructed, and are operated for the benefit of power development, both by public utilities and by private industry, and for water supply. Most of these reservoirs are small, varying from a few hundred to a few thousand acre-feet of usable storage. The total usable storage capacity on the main river above the present plant at Fifteen Mile Falls is approximately 88,300 acre-feet. This comprises practically the full control by the New England Power Association of the area above the outlet of the Connecticut Lakes.

7. Prospective future power developments.- As the most economical and easily improved head in the basin has been developed, new projects, or the development of old plants must depend in general for justification upon increased stream flow from new storage. On the main river below the Lower Fifteen Mile Falls development there are only two undeveloped sites and three existing plants that can be improved to utilize to the best advantage the head and water available. A few miles above the Lower Fifteen Mile Falls Plant known as "The Frank D. Comerford Station" is a site that has been considered for the development of an hydroelectric plant that would have a maximum head of 161 feet and an installed capacity of 125,000 kilowatts. The pond above the dam would have an effective storage capacity of about 114,000 acre-feet and would provide for re-regulating the discharge, in cooperation with the existing plants at the Lower Fifteen Mile Falls and McIndoes Falls, to give the greatest possible minimum discharge in the main river. This added storage would be of advantage to all downstream power plants, and if not full at flood times it would aid in reducing the floods in the main river below McIndoes Falls. Transmission facilities were provided at the time of constructing the Lower Fifteen Mile Falls development with a view

to the future development of the upper project. The power from the lower site is transmitted to the Boston Metropolitan Area and throughout the New England market. It now appears reasonable to expect that, with the demands for electric power on the increase, the New England power market will soon need and justify the construction of this plant. In general, however, as the greater part of the cost of producing electric power in hydroelectric plants in the fixed charges based upon the investment required for the construction of the power plants and storage reservoirs, there will be little inducement for their development unless they can be constructed to sell electric power at the cost of producing it in existing steam-power plants in Boston or other New England cities.

8. Provision for penstocks.- Studies were made of the potential development of hydroelectric power at each reservoir site in accordance with the provision of Section 5 of the Flood Control Act of 1936, H. R. 8455, "That penstocks or other similar facilities, adapted to possible future use in the development of adequate electric power may be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers."

9. Basis for providing penstocks.- An analysis was made of each site to determine the maximum possible value of power that might be developed at the site under the most favorable circumstance as produced by the following assumptions:

(1) Market available at the site at liberal values of 8 mills per kilowatt-hour for prime power and 3 mills per kilowatt-hour for secondary power.

(2) No allowance for the cost of construction or maintenance of transmission lines from the sites to a market.

(3) Construction of storage reservoirs up to their physical limits and not chargeable to power plants at site, but to downstream plants benefited by increase of low-water flow.

(4) Annual cost based upon the following charges applied only to the power station and equipment required for the development of the flow available 25% of the time on a 40% load factor:

Interest on initial cost	6.0%
Maintenance	1.0
Depreciation, obsolescence, and amortization	2.5
Taxes and insurance	1.5
Operation	1.0 plus $3,000

It is believed that at any site having an unfavorable ratio of benefits to costs, with all factors favorable to power development, no provision for penstocks should be made at this time.

10. Analysis of each site.- In Table 34 are given the essential data and results of the analysis made in accordance with the foregoing assumptions. From this table it will be noted that the ratio of the estimated annual return for power to the annual cost of production is 1.01 for Gaysville, 0.99 for Knightville, 0.83 for Newfane, and less than 0.70 for all other sites. It is emphasized that these values are dependent upon the addition of conservation storage to the storage required for flood control. Therefore, if and when conservation storage is developed, it is proposed to provide penstocks for a future power station at Gaysville, Knightville, and also at Newfane, considering the comparatively large potential storage, head, and power at this site. The sizes of penstocks recommended for these dams are given below:

Site No.	Reservoir	River	Drainage Area sq. mi.	Penstock Max.Disch. c.f.s.	Recom.Velocity ft./sec.	Recom.Diam. ft.
29A	Gaysville	White	226	1,300	10.7	12.5
40	Newfane	West	326	1,950	11.0	15.0
47	Knightville	Westfield	164	600	7.7	10.0

At the remaining reservoir sites, where penstocks are not recommended, potential power developments would be of small capacity and have a wide range in head, varying from the highest elevation reached, when the reservoir is filled, to practically zero when depleted. The initial cost of constructing the plant, with the necessary transmission lines, and the cost of operation of a plant of so small a capacity, would be large in comparison with the costs of existing power systems. Considering that under the favorable nature of the basic assumptions, power values for these sites were less than costs by more than 30 per cent, it is believed that no justification for additional provisions for penstocks could be found, regardless of changes that might be made in the basic assumptions. It has been shown that power development at the flood control sites is not economically justified under the most favorable assumptions except at Gaysville and Newfane at which sites no part of the cost of conservation could be borne by the power development. At Knightville conservation is not warranted unless a part of the cost of the reservoir is borne by a power development at the site. In general the greatest benefit from conservation storage lies in the values derived from increased low-water flows to downstream plants.

Conservation storage developed with flood control projects

11. Functions of conservation storage.- The function of conservation storage is to store or conserve water when the stream flow is

high and to release water when the stream flow is low. Its value is derived from three stages of this process: (1) By storing, it reduces the flow downstream which may or may not be great enough to cause damage. If conservation storage is not already full at flood times, there will be a value to it from reduction of flood damage; (2) By holding water in storage prior to and during the period of releasing, it provides an artificial lake which may have a recreational value; and (3) By releasing to increase the low-water flow, it may be of value in the dilution of sewage if sufficient in quantity, increases industrial water supplies, and increases the power output of run-of-river plants, both utility and industrial.

12. Power developments that would benefit from increase of low-water flow.- Reference is made to House Document No. 412, the Document 308 Report for the Connecticut River, for detailed presentation of data on existing and potential power developments in the Connecticut River Watershed. For this report, data for the plants that would be benefited by conservation storage at flood control reservoirs under consideration were abstracted, and are given in Table 35. In this Section, the comprehensive development referred to is the Comprehensive Power Development Plan in House Document No. 412.

13. Use of flood control storage for conservation.- Although a damaging flood may occur at any time during the year, it is recognized that the greatest floods are more likely to occur in March and April, or in the fall storm period from September through November. In the use of conservation storage to increase the low-water flow, water is generally stored from March to June, and released when needed during the period from June through February, with intermittent storing whenever

the flow at the downstream power plants is greater than their capacities. By regulating the rate of storing in the spring so that the volume reserved for conservation would just be filled by June 1, a part of the conservation volume would usually be available to aid in reducing a March or April flood. During the fall flood period a large part of the conservation volume would normally be available for flood control. Therefore, when considering the addition of conservation storage to that required for flood control, the following empirical rule was developed to compensate the interest providing conservation storage for the minimum flood-controlling effect that it might reasonably provide: For each inch of conservation storage provided, one-quarter inch of flood control storage may be used by the conservation interests, with the following limitations: (1) No more than three inches of flood control storage may be so used, and (2) No flood control storage less than 4.5 inches should be subject to conservation use.

14. Determination of annual power benefits.- The economic value of additional storage for conservation at the 30 flood control reservoirs that comprise the Comprehensive Plan and its alternates has been analyzed in relation to the existing power development in the Connecticut Watershed, and also to the potential comprehensive power development, which includes re-developments and new developments that may be economically justified by a more highly developed market for electric power. Eighteen of the reservoirs are limited in capacity either by the topography at the sites, or by the location above them of towns or industries which would make flowage damages prohibitive. In these cases the benefits and approximate costs were computed for an increment of capacity above that reserved for flood control and it was found definitely that conservation storage is not warranted. At the remaining 12 reservoirs the power benefits to the existing and comprehensive

developments described above were computed for one inch of storage at each reservoir, and the results are given in Table 36. In evaluating the mean annual energy benefit from the use of conservation storage, it is assumed that the storage will be used once a year, that the average per cent of plant efficiency is 80, and that the value of the energy at the switchboard is three mills per kilowatt-hour. In computing the increased prime capacity benefit from conservation storage, it is assumed that the increase in minimum discharge is equivalent to an average rate of release that will deplete one inch of storage in 150 days, and that the annual value of one kilowatt of prime capacity at the switchboard is $6. The energy benefit represents a definite saving in cost of production of steam-electric power which it would supplant, and is therefore an immediate and assured benefit. The prime power benefit, which is derived from a utilization of existing equipment, is dependent upon the future needs for additional prime capacities of the power system that it affects, because the existing capacity of steam plants, plus the prime capacity at hydro plants, with existing stream flow, is greater than that needed to meet existing load demands. As the system load increases, however, additional prime capacity will eventually be needed, and at that time the installation of steam-plant capacity can be lessened by increasing the prime capacity of hydro plants through increasing the low-water flow. The resultant saving in cost of steam installation will be directly attributable to the conservation storage used to increase the low-water flow. Although the prime capacity benefit may not be realized at present, it may become an actual benefit within a period of years if the present rate of increase of load in the Zone is maintained.

15. <u>Determination of annual cost of conservation storage.</u>- The graphs of annual cost versus reservoir capacity prepared for determination of most economical size of flood-control storage were used to determine the approximate cost of one inch of conservation storage, with slight revisions in the case of retarding basins to include cost of gates. This cost was read as the increment for one inch of capacity above the flood-control capacity. Where combined use of conservation and flood-control storage may be made under the assumption of Paragraph 13, one inch of conservation storage is actually obtained by adding only 0.8 inch of storage for conservation, in which case the cost increment is taken for 0.8 inch above the flood control capacity. The annual costs determined thusly are shown in Table 36, Column 14. For the reservoirs where conservation storage appears to be economically justified, based upon these cost data, actual estimates of annual cost of reservoirs for flood control, plus various increments of conservation storage, were made, and the annual cost of the conservation storage alone determined by subtracting from these totals the estimated annual cost for flood control alone.

16. <u>Determination of economic value of conservation storage.</u>- For both existing and comprehensive developments, the ratios of annual energy benefit to cost, and of annual energy plus power benefit to cost, were computed for one inch of storage. It may be seen from these ratios that conservation storage is economically justified at Victory and West Canaan, based upon the energy benefit to existing plants; at Groton Pond and Stocker Pond based upon the energy plus power benefit to existing plants; and at Gaysville, Ayers Brook, Perkinsville, Newfane, Priest Pond,

and Tully based upon energy plus power benefit to the Comprehensive Power Development. Detailed estimates of the annual cost of conservation storage were made for the ten sites shown above, for which it was found that conservation storage is justified under any of the four conditions evaluated. In Table 36A is given a summary of the results of the detailed analyses made to determine the economic limit of conservation capacity at the point of diminishing returns. In columns 13 and 17 is given the estimated cost per kilowatt hour of electric energy available from increased low water flows. In this table it will be noted that the ratios of annual benefits to annual costs are low, and the estimated costs of energy are high at the existing plants below Priest Pond and Tully reservoirs. This is accounted for by the lack of information in this office on the capacities of the hydraulic equipment or the power demands in the various industrial plants in this area. However, as the storage water from the reservoirs will be released during the low water flows, it appears reasonable to expect that the greater portion of these plants will be able to use the increased flows for the development of power or for processing water and that it will be a substantial benefit to them. The value of these benefits cannot be computed, and would have to be arrived at by negotiations with the Millers River industries.

17. Operation of conservation reservoirs.- The ten flood control reservoir sites where conservation storage would be economically justified if the potential power plants downstream were developed would be operated primarily for the control of floods, and, secondarily, for the benefit of the power developments below the reservoirs. Each reservoir is designed to permit regulation of the river flow to meet the requirements of any adopted operating plan. Adequate outlets are to be constructed for this purpose. This will permit the discharge or retention

of storage in the interest of flood protection or conservation. These reservoirs would operate in cooperation with the existing and any future reservoirs as a system. In this manner there can be obtained the greatest protection from floods and the greatest benefit toward regulating the flows below the reservoirs for the development of power at the existing or future plants on the main river and its tributaries.

18. Effect of operation upon power development.- The benefits from each of these ten reservoirs to power developments at each of the existing reconstructed and new plants are given in Tables 37 to 46, inclusive. An examination of these data shows that with but few exceptions the capacities of the existing hydroelectric plants on the main river are considerably in excess of primary power supply when operating on a normal load factor of 40 to 60 per cent. In Table 47 is given a summary of the power benefits to downstream plants from each of the ten conservation reservoirs at flood control dams. The present interconnection of the steam plants in the zone with the larger hydroelectric plants enables the hydro plants with excess capacity to carry during the periods of high flows, the base load of the entire system and the steam plants to carry the peaks. During the periods of low flows the steam plants carry the base loads of the system and the hydroelectric plants carry the peaks. It is shown in Paragraph 10 that no part of the cost of conservation storage can be borne by power development at the site. However, by increasing the low water flow in the Connecticut River and its tributaries, as partially regulated by existing and contemplated power utility storage developments, through the release from conservation storage at the ten flood control dams indicated in the summary table, it was found that the total estimated average increase of about 1,460 c.f.s. in low-water flow from the ten reservoirs would produce annually approximately 55 million kilowatt-hours of electric energy at the existing and 147 million kilowatt-hours at the existing and all potential future redeveloped and new plants included in the comprehensive development.

19. Redeveloped and new plants.- Should power interests desire to utilize the maximum storage that could be provided by these reservoirs, two new plants can be built on the main river, three on the White River, and three on the West River; and the existing plants at Enfield, Connecticut and Wilder, Vermont could be redeveloped. In Table 48 is given a list of possible sites for the redevelopment of existing plants and the construction of new hydroelectric stations, with the estimated total amount of electric power that will be available at each site and based upon the utilization of the regulated flows that will be available after completion of the reservoirs indicated in column 3. The existing conditions in the power industry do not warrant the construction of new hydroelectric power plants unless they can produce electric power at a cost comparable with the cost of production by the existing or future steam plants located within the zone. Further development of storage reservoirs and hydroelectric plants in the Connecticut River Basin should be on a step-by-step basis as the New England power market expands sufficiently to require added capacities of generating equipment and make existing redeveloped or new plants economically justified.

(Report continued on following page)

CONSERVATION FOR RECREATION

20. Importance of recreation.- The Connecticut River Valley is peculiarly adapted by climate and natural features for all types of recreation. For over one hundred years it has been one of the vacation areas of the United States. The automobile has brought wealth to the Valley, changing the care of summer visitors from an occasional pin-money matter to an important industry, that is growing yearly. Formerly, groups interested in recreation have been made up principally of people who spent vacations of a month or more. Today, transient visitors spending less than a day per visit, and traveling by automobile represent a large part of the total number of visitors to the Valley. The New Hampshire Planning Board shows four transient visitors came to that state in 1936 for every summer resident and gives the average transient expenditures as $1.50 per transient per day. This amount is for food and does not indicate the gross daily expenditures. The American Automobile Association shows that each visiting vacationist automobile represented an average daily total cash expenditure by its occupants of $24.50. The New England Council and the New Hampshire Planning Board indicate that during the 1936 summer season a total of 811,000 guests spent an average of about 11-1/2 days each, and that about 3-1/4 million transients spent an average of one day each, or a total of about 12 million vacation days in the Connecticut River Basin areas of the States of Vermont, New Hampshire, Massachusetts and Connecticut. These estimates are shown in the following table:

(Table on following page)

Visitors to the Connecticut River Basin - Summer, 1936

STATE	N. H. Plan. Board and N. E. Council				N.E.C. and N.H. Pl. Bd.		A.A.A.
	Guests requiring accommodations	Transients (repeaters)*	Total	A.A.A. Total	Vacation Days Excl: Transients	Vacation Days Total	Vacation Days
N. H.	294,000	1,206,000	1,500,000	211,000	3,360,000	4,566,000	3,920,00
Vt.	170,000	707,000	877,000	122,000	1,945,000	2,652,000	2,227,00
Mass.	244,000	926,000	1,150,000	156,000	2,560,000	3,486,000	2,900,00
Conn.	103,000	424,000	527,000	76,400	1,080,000	1,504,000	1,410,00
Total:	811,000	3,263,000	4,054,000	565,400	8,945,000	12,208,000	10,457,00

* A.A.A. figures do not include transient visitors.

Considering that within 300 miles of the Connecticut River Valley are located the large population centers of New York, Brooklyn, Boston, Albany, Philadelphia, Camden and Newark, that the trend towards devoting more time to recreation is causing a growing demand for increased facilities, and that most existing lakes have been developed; it is reasonable to estimate that development of additional recreation capacity can be justified.

21. Value of existing recreational facilities.- It is estimated by the New England Council that over $400,000,000 was spent by visitors to New England in the summer of 1936, of which more than $276,000,000 was spent in the four States of New Hampshire, Vermont, Massachusetts, and Connecticut. Over $50,863,000 of this amount was spent in the Connecticut River Basin. Visitors to New England used property valued at more than $500,000,000 during this same period; $63,500,000 of which was in the Connecticut River Basin. The value of facilities provided for winter sports is not included. Visitor expenditures in the Connecticut River Basin areas of the four States for 1936 are shown in the following table:

1936-Summer Visitor Expenditures
in the Connecticut River Basin

State	New England Council & N. H. Pl. Board: Estimated total expenditures		A.A.A. Estimated Total
	Exclusive of Transients	Total	
N. H.	$13,750,000	$15,550,000	$19,800,000
Vt.	7,950,000	9,000,000	11,470,000
Mass.	10,480,000	11,870,000	14,650,000
Conn.	4,840,000	5,470,000	7,170,000
Total	$37,020,000	$41,890,000	$53,090,000

22. Users of Recreation Capacity.- Conservation capacity, where allowable at flood control reservoirs, will be used for recreation purposes by three general classes of visitors: (1) by summer home owners, cottagers and campers, who not only will bring in annual income to the community by expenditures, but also, by construction of permanent buildings, development of sites for bathing, boating, and the like, will provide considerable additional taxable property; (2) the hotel and boarding house visitors especially in sections where there is now a lack of water facilities, as, for example, exists in the Bethlehem Junction area, where there is a considerable summer vacation population but recreation water facilities are inadequate; (3) the transient visitors attracted by the lake. Based on the experience in the Bethlehem Junction area, where, local sources state, more than 1,000,000 people, transients, pass per summer through Franconia Notch, the gateway to the area, it is estimated that transients will compose the major portion of the visitors.

23. Determination of reservoirs suitable for recreation conservation.- Each reservoir proposed for the Comprehensive Plan or as an alternate site was studied to determine whether additional development for recreation would jeopardize its value for flood control, whether this additional

development would be economically justified, and whether the location of the reservoir was suitable for a successful recreation development. The decision on this last was aided in several instances by marked local interest in the project. A comparison of the net annual income with the annual cost of the additional expense of providing the extra storage capacity for recreation showed whether the development for recreation was economically justified. The sites were studied for (1) topographic and shoreline conditions governing recreational facilities, and (2) as to location and accessibility.

Under facilities:- Sites were studied for:

a. possible number of cottage sites.

b. distance from water of cottages because of necessity of building outside the limits of the flood control pool area.

c. topographic features between the outer limits of the flood control pool and the smaller or conservation pool.

Under location:- Sites were studied for:

a. proximity to travel routes, to determine possible visitor traffic.

b. location in an area where the vacation public was accustomed to go.

c. the amount of use made of existing water facilities in the same section.

d. the lack of existing water facilities.

e. the proximity of the reservoir site to parks and forest preserves.

Of the 30 sites studied, 16 were located in Vermont, 8 in New Hampshire and 6 in Massachusetts. Eleven sites were found to have possibilities. Eight of the 20 reservoirs of the Comprehensive Plan were suitable for conservation capacity, and 3 of the 10 alternate sites. Of the 11 sites, 6 were in Vermont, 3 in New Hampshire and 2 in Massachusetts. At seven sites, the additional development which was justified by its power value, would also provide a lake for recreational use.

24. Discussion of sites.-

a. Bethlehem Junction - (24) - New Hampshire - is in a developed recreation area where water facilities are very much needed. There is great local desire for conservation storage. There are several large hotels and numerous smaller places, and a very determined pressure is being exercised on the local authorities to secure water facilities. Five of the large hotels are reported contemplating installing swimming pools for their guests at an average cost, estimated locally, of about $20,000 per pool. More than 1,000,000 people were counted passing through the Franconia Notch into this area in the summer of 1936. Possible annual income to the community from recreation capacity in the flood control reservoir has been estimated locally at over $400,000. There is no power value to the pool.

b. West Canaan.- (66) New Hampshire - is located on the Mascoma River near Mascoma Lake, an already developed summer recreation lake located near Lebanon, New Hampshire. This area, in which are located White River Junction, Hanover and Lebanon, is a highly developed recreation area. The White River Junction - Lebanon - Concord road passes within less than a mile of this reservoir. Visitor traffic should be large, besides which, the size of West Canaan reservoir would make it attractive for summer cottage settlement. The pond will also have a sizable annual power value.

c. Stocker Pond - (53) - New Hampshire - is situated about one mile from New Hampshire State Route No. 10, near Grantham and about eight miles from Lake Sunapee, already heavily developed as a recreation center. It is about twenty-three miles by automobile from U. S. Highway No. 5, the main north and south artery. Visitor traffic at Stocker Pond should be high because of its location. It will have a sizable shoreline available for real estate development and the pond will have power value.

d. Victory - (22) - Vermont - is an undeveloped site at present, but has power and recreation possibilities. It is located about eighteen miles northeast of St. Johnsbury, Vermont, and about four miles from the through route, Burlington, Vermont - Bangor, Maine. Traffic moving through the Upper Connecticut Valley as well as cross traffic passes this site. It is within seven miles of Darling State Forest Reservation and about thirty-five miles by automobile from the popular recreation section centered about Bethlehem Junction. Victory would provide recreational facilities for through visitor traffic as well as for St. Johnsbury, which is at present served by the inadequate facilities of the Sleeper's River.

e. Groton Pond - (27) - Vermont - This site is at present developed and is included in Table 49a to indicate possible recreation and power benefits which might act to lessen construction and damage costs. It is located midway between the Montpelier - St. Johnsbury, and the Montpelier - Woodsville routes, in the Groton State Forest.

f. Union Village - (48) - Vermont - There is considerable local interest in recreation facilities. The site is less than five miles from U. S. Highway No. 5, the main route north and south through the Connecticut Valley. It is about fifteen miles by automobile from Hanover, a recreation center. With proper facilities, visitor traffic should be large. There is no power value.

g. Ayers Brook - (30) - Vermont - is less than three miles from Allis State Forest Park and thirty miles from Montpelier. It is somewhat off the main travel routes and could not be expected to develop the visitor traffic of more favorably located sites. Some power capacity is available.

h. Gaysville - (29) - Vermont - is in the same section as Ayers Brook. It is on the edge of the Green Mountain National Forest and is about ten miles from the through route Glen Falls - Rutland - White River Junction. Gaysville should enjoy a better traffic density than Ayers Brook because it is nearer a main traffic artery and because its size will make it attractive for recreation, even in an area provided with lakes. It is suitable for power.

i. Newfane - (40) - Vermont - This site, located partly in Townshend State Forest, will have a comparatively large lake. While the number of cottage sites is large, their value, owing to distance from probable recreation shoreline, is low. This reservoir will attract some visitor traffic, being only fifteen miles from Brattleboro and ten miles from U. S. Highway No. 5 by way of Dummerston, but unless the size of pond available for recreation can be increased to reduce the distance between the flood control and recreation pond shorelines, its recreation value will be limited. There will be a considerable power value to the site.

j. Tully - 62A - Massachusetts - is located three miles north of the Troy - Fitchburg - Boston road passing through Athol. This is a well traveled route with several popular vacation centers nearby. Visitor traffic through this area should be fairly high and permit a substantial annual recreation income. The development is suitable for power.

k. Priest Pond - 61A - Massachusetts - is located in the same area within a mile of the Tully Reservoir and would benefit from the same routes of traffic as Tully but owing to small size of lake will not permit as large a development. The lake is suitable for power.

25. Recreation Income.- Sources of recreation income for the Connecticut River Basin are indicated in Table 49b, based on data

furnished by the New Hampshire Planning Board. Recreation income to a reservoir community is composed of that from the summer cottagers, and other resident vacationists, and that from transient visitors. Net income to the community per cottage per season, exclusive of taxes, was taken from available statistics on lake resorts in New Hampshire, from which the indicated mean average was found to be about $350. In figuring possible income from transients, data offered at one of the developed recreation areas indicated that about 25% of transient visitors to a reservoir site would probably make expenditures and that the major portion of these expenditures would be for food. The average net income per day from such expenditure per transient was given as about 40%. Total recreation use of an area has been estimated to be an average of approximately 2,990 vacation days per cottage, for resident and transient visitors combined, from which the total net income per cottage is estimated at approximately $1,438 per season. On the basis that cottage development is generally an indication of the popularity of a recreation site, the total income to a community has been estimated on the basis of the number of cottages in the prospective development.

26. Conclusions.- Development of conservation capacity at private expense appears justified at 11 sites. The estimated recreation income at the 11 sites is indicated in Table 49. A comparison of the annual benefits and costs of the flood control, power, and recreation features of the 11 sites is made in Table 49a.

SECTION I

TABLE REFERENCE

TABLE I RAINFALL STATIONS

STATION: TOWN OR CITY	STATION: STATE	OPERATED BY	TIME OF READING	LENGTH OF RECORD IN YEARS
Ansonia	Conn.	Ansonia Water Co.	A	
Bakersville	"	Hartford Met. W. S.	A	
Baltic	"	U.S.W.B. (Sup)	B	26
Barkhamstead	"	U.S.W.B. (Sup)	B	13
Bridgeport	"	U.S.W.B.	B	43
Bristol Center	"	Bristol Water Co.	A	
Burlington	"	U.S.W.B. (Sup.)	B	18
Colchester	"	U.S.W.B.	B	45
Cream Hill	"	U.S.W.B.	B	41
Dawson Res.	"	New Haven Water Co.	A	
Derby-Shelton	"	Birmingham Water Co.	A	
East Hartland	"	U.S.W.B.	A	20
Easton Lake Res.	"	Bridgeport Hydraulic Co.		
Enfield	"	Northern Ct. Power Co.	A	
Falls Village	"	U.S.W.B. (Sup)	B	46
Groton	"	Groton Water Co.	A	
Hartford	"	U.S.W.B.	C	77
Hemlocks Res.	"	Bridgeport Hydraulic Co.		
Howard Res.	"	Manchester Water Co.		
Laurel Res.	"	Stamford Water Co.	A	
Meade Pond Res.	"	Stamford Water Co.	A	
Middletown	"			
Milford Res.-	"	New Haven Water Co.	A	
Mt. Carmel	"	U.S.W.B. (Sup)	B	1
Mt. Rigby Res.	"	Middletown Water Co.		
Naugatuck	"	Naugatuck Water Co.	A	
Nepaug Dam	"	Hartford Met. W.S.		
New Hartford	"	U.S.W.B. (Sup)	B	
New Haven	"	U.S.W.B.	C	93
New London	"	U.S.W.B.	B	66
Norfolk-West	"	Edward C. Childs	A	
N. Branford Res.	"	New Haven Water Co.	A	
N. Grosvenordale	"			
N. Guilford Res.	"	New Haven Water Co.	A	
N. Stamford	"	Stamford Water Co.	A	
Norwalk	"	U.S.W.B.	A	45
Old Greenwich	"	U.S.W.B. (Sup)	B	4
Phelps Brook Res.	"	Hartford Met. W.S.	A	
Prospect Res.	"	New Haven Water Co.	A	
Putnam Res.	"	Greenwich Water Co.	A	1
Reservoir No. 4	"	Hartford Met. W.S.		
Reservoir No. 6	"	Hartford Met. W.S.		
Roaring Brook Res.	"	Manchester Water Co.		
Salisbury	"	U.S.W.B. (Sup)	B	3
Saltonstall Res.	"	New Haven Water Co.	A	
Shuttle Meadow Res.	"	New Britain Water Co.	A	
Storrs	"	U.S.W.B.	B	44
Thomaston Res.	"	Waterbury Water Co.		
Thompsonville	"			
Torrington	"	Torrington Register	A	
"	"	Gen. S. H. Wadhams	A	
"	"	Torrington Water Co.		
Trap Falls Res.	"	Bridgeport Hydraulic Co.		
Waterbury	"	U.S.W.B.	B	
Waterbury	"	City of Waterbury		
Wepawaug Res.	"	New Haven Water Co.	A	
West Cornwall	"			
W. Hartford Res. No. 1	"	Hartford Met. W.S.	A	
W. Hartland	"	U.S.W.B. (Sup)	B	12
W. Hill	"	Hartford Met. W.S.		
Whigville Res.	"	New Britain Water Co.	A	
Whitney Res.	"	New Haven Water Co.	A	
Wilton	"	Norwalk Water Co.	A	
Wolcott Res.	"	New Britain Water Co.	A	
Adams	Mass.	U.S.W.B. (Sup)-N.E.P.	A	11
Amherst	"	U.S.W.B.	A	101
Ashby	"	M.S.D.P.H.	A	
Ashland	"	U.S.W.B. (Sup) (Y)	A	51
Athol	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	
Athol-Fryville	"	M.S.D.P.H.	A	
Attleboro	"	M.S.D.P.H. (Pumping Sta.)	A	
Baldwinville	"	M.S.D.P.H.	A	
Barre	"	M.S.D.P.H.	A	
Beechwood	"	M.S.D.P.H.	A	
Beverly (Wenham Lake)	"	M.S.D.P.H.	A	
Blandford	"	M.S.D.P.H.	A	
Blue Hill	"	U.S.W.B.	B	51
Bondsville	"	M.S.D.P.H.	A	
Boston	"	U.S.W.B.	C	119
Boylston	"	U.S.W.B. (Sup) (Y)	B	41
Bridgewater	"	M.S.D.P.H.	A	
Brockton	"	U.S.W.B. (Sup)	B	43
Brockton	"	M.S.D.P.H. (Sewage Works)	A	
Cambridge	"	City Engineer		
Charlton	"	M.S.D.P.H.	A	
Chatham	"	M.S.D.P.H.	A	
Chester	"	M.S.D.P.H.-U.S.W.B. (Sup.)	A	24
Chesterfield	"	M.S.D.P.H.- U.S.W.B.	A	18
Chestnut Hill	"	U.S.W.B. (Sup) (Y)	A	64
Chicopee Falls	"	M.S.D.P.H.	A	
Clinton	"	U.S.W.B. (Y)	A	59
Colrain	"	M.S.D.P.H.	A	
Concord	"	U.S.W.B.	B	49
Cummington	"	M.S.D.P.H.	A	
Dalton	"	Power Co.		
Dana	"	M.S.D.P.H.	A	
Dana (North)	"	M.S.D.P.H.	A	
E. Northfield	"	M.S.D.P.H.	A	
E. Pepperel	"	M.S.D.P.H.	A	
E. Wareham	"	U.S.W.B.	A	27
Edgartown	"			
Egremont	"	M.S.D.P.H.-U.S.W.B.	A	32
Enfield	"	(Y)	A	
Fall River	"	U.S.W.B.	B	64
Falmouth	"	M.S.D.P.H.-U.S.W.B.	A	39
Fitchburg	"	U.S.W.B.	A	72
Framingham	"	U.S.W.B. (Y)	A	61
Franklin	"	M.S.D.P.H.-U.S.W.B.	A	15
Gardner	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	31
Gloucester	"	U.S.W.B. (Sup)	B	22
Granville Dam	"	M.S.D.P.H.	A	
Greenbush	"	M.S.D.P.H.	A	
Greenfield	"	M.S.D.P.H.	A	16
Greenwich	"	M.S.D.P.H.	A	
Groton	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	49
Hardwick	"	M.S.D.P.H.- U.S.W.B.	A	17
Haverhill	"	U.S.W.B. (City Engineer)	B	42
Heath	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	17
Hingham	"	Town		
Holyoke	"	U.S.W.B.		30
Holyoke	"	Whiting Street		
Holyoke	"	Ashley Ponds		
Holyoke	"	High Service		
Hoosac Tunnel	"	U.S.W.B. (Sup)-N.E.P.	A	7
Housatonic	"			
Hubbardston	"	M.S.D.P.H.- U.S.W.B.	A	16
Huntington	"	M.S.D.P.H.	A	
Hyannis	"	U.S.W.B.	B	44

STATION: TOWN OR CITY	STATION: STATE	OPERATED BY	TIME OF READING	LENGTH OF RECORD IN YEARS
Ipswich	Mass.	M.S.D.P.H.- U.S.W.B.	A	26
Jamaica Plains	"	M.S.D.P.H.	A	
Jefferson	"	U.S.W.B. (Sup Y)	A	40
Kenoza Lake	"	U.S.W.B. (Sup Y)	A	
Lake Cochituate	"	U.S.W.B. (Sup Y)	A	85
Lakeville	"	M.S.D.P.H.	A	
Lawrence	"	U.S.W.B. (Sup)	A	81
Lawrence	"	M.S.D.P.H. (Experiment Sta.)	A	
Leominster	"			52
Lincoln	"	Hobbs Brook Res.		
Littleton	"	M.S.D.P.H.	A	
Lowell	"	U.S.W.B.	A	111
Ludlow	"	M.S.D.P.H.	A	
Lynn	"	M.S.D.P.H.	A	
Manchester	"	M.S.D.P.H.	A	
Mansfield	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	26
Marlboro	"	Water Works		
Middleboro	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	51
Middlefield	"	M.S.D.P.H.	A	
Middleton	"	U.S.W.B. (Sup)	A	11
Milford	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	22
Millbury	"	U.S.W.B. (Sup) N.E.P.	A	7
Millis	"	M.S.D.P.H.	A	
Monroe Bridge	"	N.E.P.		
Monson	"	M.S.D.P.H.	A	
Nantucket	"	U.S.W.B.	C	63
Natick	"			
New Bedford	"	U.S.W.B. (City Engineer)	B	23
New Bedford	"	M.S.D.P.H. (L. J. Hathaway)	A	
New Braintree	"	M.S.D.P.H.	A	
Newburyport	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	
New Salem	"	M.S.D.P.H.	A	
Newton	"	M.S.D.P.H.	A	
N. Adams (Broad Brook)	"			
N. Adams (Notch Brook)	"			
N. Andover	"	M.S.D.P.H.	A	
Northbridge	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	65
Norwood	"	M.S.D.P.H.	A	
Otis	"	M.S.D.P.H.- U.S.W.B.	A	23
Otis (West)	"	M.S.D.P.H.	A	
Peabody (West)	"	M.S.D.P.H.	A	
Pelham	"	M.S.D.P.H.	A	
Pembroke	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	21
Peru	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	24
Petersham	"	U.S.W.B. (Sup) (Y)	A	24
Pittsfield	"	U.S.W.B.	A	36
Packardsville	"	M.S.D.P.H.	A	
Phillipston	"	M.S.D.P.H.	A	
Plainfield	"	M.S.D.P.H.- U.S.W.B. (Sup)	A	18
Plymouth	"	U.S.W.B.	A	50
Prescott	"	M.S.D.P.H.	A	
Princeton	"	U.S.W.B. (Sup) (Y)	A	49
Provincetown	"	U.S.W.B.	B	50
Reading	"			
Rochester	"			
Rockport	"	U.S.W.B.	A	35
Rutland (North)	"	M.S.D.P.H.	A	
Rutland	"	M.S.D.P.H.	A	
Salem	"	M.S.D.P.H.	A	
Savoy	"	N.E.P.		
Scituate	"	M.S.D.P.H.	A	
Shelburne Falls	"	U.S.W.B. (Sup)-N.E.P.	A	23
Shelburne Falls	"	M.S.D.P.H.	A	
Shutesbury	"	M.S.D.P.H.	A	
Southampton	"	Former Res.		
Southboro	"	Sudbury Dam (Y)		
Southboro	"	Cordaville (Y)		
Southbridge	"	M.S.D.P.H.- U.S.W.B.	A	25
S. Deerfield	"	M.S.D.P.H.	A	
Spot Pond	"	U.S.W.B. (Sup) (Y)	A	38
Springfield	"	U.S.W.B.	B	89
Springfield	"	U.S.W.B.	B	
Springfield	"	U.S. Arsenal		
Springfield	"	City Hall		
Springfield	"	West Parish Filters		
Springfield	"	Provin Mt. Res.		
State Farm	"	Bridgewater		
Sterling	"	U.S.W.B. (Sup) (Y)	A	40
Stockbridge	"	U.S.W.B. (Sup)	B	16
Swampscott	"	U.S.W.B. (Sup)	B	9
Taunton	"	U.S.W.B. (Water Works)	A	62
Turner's Falls	"	U.S.W.B.	A	41
Walpole (East)	"	M.S.D.P.H.	A	
Waltham	"	M.S.D.P.H. (Water Works)	A	
Ware	"	M.S.D.P.H.-U.S.W.B. (Sup)	A	18
Ware (West)	"	M.S.D.P.H.	A	
Wareham	"	M.S.D.P.H.	A	
Wareham (East)	"			
Ware R. Intake	"	(Y)	A	
Warren	"	M.S.D.P.H.	A	
Warwick	"	M.S.D.P.H.	A	
Webster	"	M.S.D.P.H.	A	
Wendell	"	M.S.D.P.H.	A	
West Brookfield	"	M.S.D.P.H.	A	
Westfield	"	U.S.W.B. (Sup)	B	31
Westfield	"	M.S.D.P.H.	A	
Westhampton	"	White Res.		
Westminster	"	Wachusett Lake		
Westminster	"	Meetinghouse Pond		
Weston	"	Stony Brook Res.		47
Weston College	"	U.S.W.B. (Sup)	B	
West Roxbury	"	M.S.D.P.H. (Brookline Pump Sta.)	A	
West Rutland	"	U.S.W.B. (Sup) (Y)	A	9
Williamsburg	"	M.S.D.P.H.	A	
Williamstown	"	U.S.W.B.	B	76
Williamsville	"	M.S.D.P.H.	A	
Wilmington	"	M.S.D.P.H.	A	
Winchendon	"	M.S.D.P.H.- U.S.W.B.	A	44
Winchester	"	M.S.D.P.H.	A	
Wollaston	"	M.S.D.P.H.	A	
Worcester-Winter Hill	"	U.S.W.B.	B	80
Worcester- Clark V.	"	U.S.W.B. (Sup)	A	
Worcester	"	Clinton Dam	A	
Worcester	"	M.S.D.P.H.- Kendall Res.	A	
Worcester	"	Kettle Brook Res.	A	
Worcester	"	Lynde Brook Res.	A	
Worcester	"	Holden Res. No. 2	A	
Worthingdon	"	M.S.D.P.H.	A	
Wrentham	"	M.S.D.P.H.	A	
Eustis	Maine	U.S.W.B.	B	23
Hiram	"	U.S.W.B. (Sup)	B	12
North Bridgeton	"	U.S.W.B.	B	44
Portland	"	U.S.W.B.	C	70
Rumford	"	U.S.W.B.	B	43
Berlin	N.H.	U.S.W.B.	A	37
Bethlehem	"	U.S.W.B. (Sup)	A	44
Bethlehem	"	N.E.P.		
Claremont	"			
Concord	"	U.S.W.B. (Sup)	A	84
Dixville	"	U.S.W.B. (Sup)	A	6

STATION: TOWN OR CITY	STATION: STATE	OPERATED BY	TIME OF READING	LENGTH OF RECORD IN YEARS
Dummer	N.H.			
Durham	"	U.S.W.B.	B	44
Errol	"	U.S.W.B. (Sup)	B	51
First Conn. Lake	"	U.S.W.B.	B	19
Fitzwilliam	"	U.S.W.B. (Sup)	A	17
Franklin	"	U.S.W.B.	B	35
Garvins Falls	"			
Glenn Cliff	"	U.S.W.B. (Sup)	A	28
Gorham	"			
Greggs Falls	"			
Hampton Beach	"	U.S.W.B. (Sup)	B	8
Hanover	"	U.S.W.B.	B	96
Keene	"	U.S.W.B.	B	7
Lakeport	"	U.S.W.B. (Sup)	A	80
Lancaster	"	U.S.W.B. (Sup)	B	
Lincoln	"	U.S.W.B.	A	16
Littleton	"			
Manchester	"	U.S.W.B. (Sup)	B	62
Merry Meeting L	"			
Milan	"	U.S.W.B. (Sup)	B	22
Mt. Washington	"	U.S.W.B. (Sup)	B	
Nashua	"	U.S.W.B.	A	53
Nashua	"	Penn. Waterworks		
Nashua	"	Jackson Mills		
New Durham	"	U.S.W.B.	A	12
Newport	"	U.S.W.B.	A	8
No. Stratford	"			
Pinkham Notch	"	U.S.W.B. (Sup)	B	7
Pittsburg (1st Lake)	"			
Pittsburg (2nd Lake)	"			
Plymouth	"	U.S.W.B.	B	49
Plymouth	"			
Pontocook Dam	"			
Randolph	"		B	
Twin Mt.	"			
West Stewartson	"			23
Wolfboro Falls	"	U.S.W.B.	B	10
Woodsville	"			
York Pond	"	U.S.W.B.	B	8
Albany	N.Y.	U.S.W.B.	C	111
Bedford Hills	"	U.S.W.B.	B	47
Cairo	"	U.S.W.B.	B	13
Canton	"	U.S.W.B.	C	
Carmel	"	U.S.W.B.	A	46
Chazy	"	U.S.W.B.	B	
Conklingville	"	U.S.W.B.	B	13
Dannemora	"	U.S.W.B.	B	
Glens Falls	"			
Harkness	"			
Glenham	"	U.S.W.B.	A	5
Greenfield Center	"	U.S.W.B.	B	39
Lake Placid Club	"	U.S.W.B.	A	
Mechanicville	"	U.S.W.B.	A	26
Mt. McGregor	"	U.S.W.B.	B	23
New York	"	U.S.W.B.	C	111
Poughkeepsie	"	U.S.W.B.	A	33
Rifton	"	U.S.W.B.	B	15
Spier Falls	"	U.S.W.B.	B	35
Voorheesville	"	U.S.W.B.	A	21
Wappingers Falls	"	U.S.W.B.	B	47
West Point	"	U.S.W.B.	B	85
Willsboro	"	U.S.W.B.	B	
Brome	Quebec	Meteorological Ser.	A	
Farnham	"	" "	A & B	
Lennoxville	"	" "	A & B	
Sherbrooke	"	" "	A & B	
Block Island	R.I.	U.S.W.B.	C	59
Fiskville	"	Providence Water Supply		
Hopkins Mills	"	Providence Water Supply		
Kent	"	Providence Water Supply		
Kingston	"	U.S.W.B.	B	48
Newport	"			
No. Scituate	"	Providence Water Supply		
Pawtucket	"	U.S.W.B. (Sup)	B	36
Pawtucket	"	Diamond Hill		
Pawtucket	"	Pumping Sta. No. 3		
Pawtucket	"	Masonic Building		
Providence	"	U.S.W.B.	C	105
Providence	"	Sockanosset		
Providence	"	Pettaconset Res.		
Providence	"	Precipitation Plant		
Providence	"	Fruit Hill Res.		
Providence	"	Hope Res.		
Rocky Hill	"	Providence Water Supply		
Wakefield	"			
Westerly	"			
Westerly	"	Water Works		
Woonsocket	"			
Bellows Falls	Vermont	U.S.W.B. (Sup)	A	35
Bennington	"	U.S.W.B. (Sup)	B	23
Bloomfield	"	U.S.W.B.- N.E.P.	B	31
Brattleboro	"	U.S.W.B.- N.E.P.	A	42
Burlington	"	U.S.W.B.	C	103
Cavendish	"	U.S.W.B.- N.E.P.	B	34
Chelsea	"	U.S.W.B.- N.E.P.	B	51
Chittenden	"	Vt. Hydroelectric Co.		
Cornwall	"	U.S.W.B.	B	50
East Barnet	"	U.S.W.B. (Sup)- N.E.P.	A	7
East Ryegate	"	U.S.W.B. (Sup)- N.E.P.	A	14
Enosburg Falls	"	U.S.W.B.	B	46
Garfield	"			
Gilman	"	U.S.W.B. (Sup)	A	7
MacIndoes Falls	"	U.S.W.B. (Sup)	A	
Mays Mills	"	U.S.W.B. (Sup)-N.E.P.	A	7
Middlebury	"			
Molly's Falls	"			
Newfane	"	U.S.W.B. (Sup)-N.E.P.	B	24
Newport	"	U.S.W.B.	B	7
Northfield	"	U.S.W.B.	C	43
Readsboro	"	U.S.W.B. (Sup)-N.E.P.	A	15
Rochester	"	N.E.P.	A	8
Rutland	"	U.S.W.B.	B	21
Rutland	"	Vt. Hydroelectric Co.		
Searsburg	"	N.E.P.		
Searsburg Mt.	"	U.S.W.B. (Sup)-N.E.P.	A	12
Searsburg Sta.	"	U.S.W.B. (Sup)-N.E.P.	A	13
Sherman	"			
Silverlake	"	Vt. Hydroelectric Co.		
Somerset	"	U.S.W.B. (Sup)	A	25
So. Londonderry	"	N.E.P.		
St. Johnsbury	"	U.S.W.B. - N.E.P.	B	43
Vernon	"	N.E.P.	A	40
W. Burke	"	U.S.W.B.- N.E.P.	A	7
W. Hartford	"	U.S.W.B. (Sup)	B	7
White R. Junct.	"	U.S.W.B. (Sup)-N.E.P.	A	23
Whitingham	"	U.S.W.B. (Sup)-N.E.P.	A	14
Wilder	"	U.S.W.B. (Sup)	A	17
Wilmington	"	U.S.W.B. (Sup)-N.E.P.	A	16
Woodstock	"	U.S.W.B.- N.E.P.	B	69

Legend:
A – 8 A.M.
B – 6 P.M.
C – Continuous Recording

U.S.W.B. – Weather Bureau
U.S.W.B. (Sup) – Weather Bureau Supplemental
M.S.D.P.H. – Massachusetts State Dept. Public Health

N.E.P. – New England Power Co.
(Y) – Metropolitan District Water Supply Commission

CT-9-1077

TABLE 2

STREAM GAGING STATIONS

EXISTING STREAM-GAGING STATIONS

RIVER	GAGING STATIONS	DRAINAGE AREA SQUARE MILES	TYPE OF GAGE	DATE ESTABLISHED	DATE DISCONTINUED	DATE REESTABLISHED
Connecticut	First Connecticut Lake near Pittsburg, N.H.	83.0	Recording	Apr. 1927		
Connecticut	At North Stratford, N.H.	796.0	"	Aug. 1930		
Connecticut	At Dalton, N.H.	1538.0	"	Oct. 1935		
Connecticut	At South Newbury, Vt.	2825.0	"	July 1918		
Connecticut	At White River Junction, Vt.	4068.0	"	Oct. 1911		
Connecticut	At Turners Falls, Mass.	7138.0	"	Jan. 1915		
Connecticut	At Montague City, Mass.	7840.0	"	Oct. 1929		
Connecticut	At Thompsonville, Conn.	9637.0	"	July 1928		
Passumpsic	At Passumpsic, Vt.	423.0	"	Nov. 1928		
Moose	At St. Johnsbury, Vt.	112.0	Chain	Aug. 1928		
Ammonoosuc	Near Bath, N.H.	393.0	Recording	Oct. 1935		
White	Near Bethel, Vt.	241.0	"	June 1931		
White	At West Hartford, Vt.	690.0	"	June 1915		
Mascoma	At Mascoma, N.H.	153.0	"	Aug. 1923		
Ottauquechee	At North Hartland, Vt.	221.0	"	Oct. 1930		
Sugar	At West Claremont, N.H.	269.0	"	Oct. 1928		
Black	At North Springfield, Vt.	158.0	"	Nov. 1929		
West	At Newfane, Vt.	308.0	"	Sept. 1919	Sept. 1923	Oct. 1928
Ashuelot	Near Gilsum, N.H.	71.1	"	Aug. 1922		
Ashuelot	At Hinsdale, N.H.	420.0	"	Feb. 1907	Dec. 1909	July 1914
Otter	Near Keene, N.H.	41.8	"	Oct. 1923		
South Branch - Ashuelot	At Webb near Marlboro, N.H.	36.6	"	Nov. 1920		
Millers	Near Winchendon, Mass.	80.0	"	July 1916		
Millers	At Erving, Mass.	370.0	"	Aug. 1914		
Sip Pond Brook	Near Winchendon, Mass.	19.0	"	May 1916		
Priest Brook	Near Winchendon, Mass.	18.0	Float	May 1916		
East Branch - Tully	Near Athol, Mass.	49.9	Staff	June 1916		
Moss Brook	At Wendell Depot, Mass.	12.2	"	June 1909	Aug. 1910	June 1916
Deerfield	At Charlemont, Mass.	362.0	Recording	June 1913		
Ware	At Cold Brook, Mass.	98.2	"	Jan. 1928		
Ware	At Gibbs Crossing, Mass.	199.0	"	Aug. 1912		
Chicopee	At Bircham Bend, Mass.	703.0	"	Aug. 1928		
Swift	At West Ware, Mass.	186.0	"	July 1910		
Quaboag	At West Brimfield, Mass.	151.0	"	Aug. 1909		
Westfield	At Knightville, Mass.	162.0	Chain	Aug. 1909		
Westfield	Near Westfield, Mass.	497.0	Recording	June 1914		
Middle Branch - Westfield	At Goss Heights, Mass.	53.0	"	July 1910		
Westfield Little	Near Westfield, Mass.	48.5	"	July 1905		
Scantic	At Broad Brook, Conn.	98.7	"	Aug. 1928		
Farmington	Near New Boston, Mass.	92.0	"	May 1913		
Farmington	At Riverton, Conn.	217.0	"	Sept. 1929		
Farmington	At Tariffville, Conn.	569.0	"	Aug. 1928		
Burlington Brook	Near Burlington, Conn.	4.1	"	Sept. 1931		
Hockanum	Near East Hartford, Conn.	75.5	"	Sept. 1919	Sept. 1921	July 1928
Salmon	Near East Hampton, Conn.	105.0	"	July 1928		

DISCONTINUED STREAM-GAGING STATIONS

RIVER	GAGING STATIONS	DRAINAGE AREA SQUARE MILES	TYPE OF GAGE	DATE ESTABLISHED	DATE DISCONTINUED	DATE REESTABLISHED
Connecticut	Waterford, Vt.	1604.0	Chain	Aug. 1930	Sept. 1935	
Connecticut	Orford, N.H.	3100.0	Chain & Staff	Aug. 1900	Sept. 1921	
Connecticut	Sunderland, Mass.	7890.0	Chain & Recording	Mar. 1904	Sept. 1932	
Connecticut	Holyoke, Mass.	8284.0	Float	Jan. 1880	Dec. 1899	
Connecticut	* Hartford, Conn.		Staff & Recording	Feb. 1896	Dec. 1908	
Israel	Jefferson Highlands, N.H.	8.7	Chain	Sept. 1903	Oct. 1906	
Israel	Jefferson Highlands, N.H.	21.2	"	Sept. 1903	May 1907	
Passumpsic	Pierce's Mills, Vt.	237.0	Staff	May 1909	July 1919	
Passumpsic	* St. Johnsbury Center, Vt.	244.0	Chain	June 1903	Nov. 1903	
Ammonoosuc	Bretton Woods, N.H.	34.0	"	Aug. 1903	Apr. 1907	
Zealand	Twin Mountain, N.H.	14.0	"	Aug. 1903	Apr. 1907	
Little	* Twin Mountain, N.H.	11.0	"	Jan. 1904	Sept. 1905	
White	Sharon, Vt.	643.0	Staff & Chain	July 1903	Nov. 1904	
Second Branch - White	North Randolph, Vt.	27.0	Staff	Oct. 1920	Sept. 1921	
Ottauquechee	Woodstock, Vt.	126.0	Chain	June 1928	Sept. 1930	
Sugar	Claremont, N.H.	258.0	"	May 1928	Dec. 1928	
Ashuelot	* Winchester, N.H.	385.0	"	July 1903	Nov. 1904	
Minnewawa Brook	Marlboro, N.H.	31.7	Recording	July 1919	Mar. 1922	
Pratt Brook	Chesham, N.H.	11.2	Staff	July 1919	Sept. 1921	
Millers	Wendell Depot, Mass.	354.0	Chain	June 1909	Sept. 1913	
Otter	Gardner, Mass.	20.0	Staff	June 1916	Sept. 1917	
Deerfield	Hoosac Tunnel, Mass.	257.0	Chain	Aug. 1909	Nov. 1913	
Deerfield	Shelburne Falls, Mass.	501.0	Staff	June 1907	Dec. 1913	
Deerfield	* Deerfield, Mass.	550.0	Chain	Mar. 1904	Dec. 1905	
Ware	* Ware, Mass.	162.0	"	Sept. 1904	Dec. 1909	
Burnshirt	Templeton, Mass.	8.4	Staff	May 1909	Dec. 1909	
Quaboag	West Warren, Mass.	144.0	"	Oct. 1904	May 1907	
Westfield	* Russell, Mass.	331.0	Chain	Apr. 1904	Dec. 1905	
West Branch - Westfield	* Chester, Mass.	73.0	"	Sept. 1915	Dec. 1915	
Borden Brook	Westfield, Mass.	8.0	Float	Jan. 1910	Sept. 1918	
Westfield Little River	Near Blandford, Mass.	43.0	Staff & Chain	July 1905	Dec. 1909	
Salmon	Leesville, Conn.	115.0	Staff	Apr. 1905	Mar. 1906	

* Gage-height readings only.

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TABLE 3 Volume and Peak Discharges of Floods of November 1927 and March 1936.

River	Station	Drainage Area	1927 Rain Fall	1927 Run Off	1927 Peak Discharge	1936 Rain Fall	1936 Run Off From Rain	1936 Run Off From Snow	1936 Run Off Total	1936 Peak Discharge
		Sq.Mi.	Inches	Inches	C.F.S.	Inches	Inches	Inches	Inches	C.F.S.
Connecticut	Dalton	1,538				4.40	4.40	2.71	7.11	48,200
Connecticut	Waterford	1,604	5.25	3.00	30,200					
Passumpsic	Passumpsic	423	5.69	4.10	25,200	3.23	3.12	3.31	6.43	16,000
Wells	At the mouth	99	6.51	5.15						
Ammonoosuc	Bath	393	5.78	4.92	37,600	4.10	3.60	4.66	8.26	27,900
	Reach #1 Local	306	5.63	4.15						
	Reach #1 Local	471				3.66	3.66	4.12	7.78	
Connecticut	South Newbury	2,825	5.19	3.64	65,900	3.95	3.95	3.33	7.28	77,800
Waits	At the mouth	156	6.74	5.05						
Ompomponoosuc	At the mouth	136	7.07	5.44						
White	West Hartford	690	7.81	6.20	70,300	2.59	2.59	5.37	7.96	37,000
	Reach #2 Local	261	6.55	4.80						
	Reach #2 Local	553				3.77	3.77	3.87	7.64	
Connecticut	White River Jct.	4,068	5.84	4.26	136,000	3.68	3.68	3.77	7.45	120,000
Mascoma	Mascoma	153	6.25	2.37		3.85	3.85	2.85	6.70	4,840
Ottauquechee	North Hartland	221	7.49	5.92	30,400	3.48	3.45	5.53	9.01	19,200
Sugar	West Claremont	269	4.75	2.53	9,400	3.62	3.62	3.10	6.72	14,000
Black	North Springfield	158	7.85	6.37	20,500	3.74	3.74	3.00	6.74	14,700
	Reach #3 Local	518	5.80	3.80		3.87	3.87	2.80	6.67	
Connecticut	Bellow Falls	5,387	5.93		150,500				7.36	171,000
Saxtons	At the mouth	78	6.57	4.77						
West	Newfane	308	8.60	7.35	53,100	4.64	4.71	4.16	8.87	39,000
	Reach #4 Local	466	5.18	3.03						
	Reach #4 Local	544				4.94	4.94	4.00	8.94	
Connecticut	Vernon	6,239	6.02	4.28	159,000	3.85	3.85	3.71	7.56	198,500
Ashuelot	Hinsdale	420	4.94	2.70	6,700	4.26	4.26	3.07	7.33	16,600
Millers	Erving	370	4.68	2.55	5,600	4.85	4.85	3.65	8.50	19,700
Deerfield	Charlemont	362	7.50	3.83	16,800	4.72	4.72	5.28	10.00	28,400
	Reach #5 Local	449				4.53	4.53	4.50	9.03	
	Reach #5 Local	509	4.41	2.21						

TABLE 3 (Cont.) Volume and Peak Discharges of Floods of November 1927 and March 1936.

River	Station	Drainage Area	1927			1936				
			Rain Fall	Run Off	Peak Discharge	Rain Fall	Run Off From Rain	Run Off From Snow	Run Off Total	Peak Discharge
		Sq.Mi.	Inches	Inches	C.F.S.	Inches	Inches	Inches	Inches	C.F.S.
Connecticut	Montague City	7,840			188,000	3.98	3.98	3.62	7.60	247,000
Connecticut	Sunderland	7,900	5.92	3.95						
Chicopee	Bircham Bend	703	4.17	2.00		5.41	5.41	0.35	5.76	20,300
Westfield	Westfield	497	6.16	4.72	42,500	4.83	4.83	3.17	8.00	48,200
	Reach #6 Local	537	4.67	2.59						
	Reach #6 Local	597				4.05	4.05	1.34	5.39	
Connecticut	Thompsonville	9,637	5.74		202,000	4.40	4.40	2.92	7.32	282,000
Scantic	Broad Brook	99	3.46	1.48		4.24	4.24	1.05	5.29	2,800
Farmington	Tarriffville	569	5.98	4.02		3.97	3.97	1.13	5.10	22,200
	Reach #7 Local	297	3.75	1.66		3.17	3.17	1.13	4.30	
Connecticut	Hartford	10,602	5.66	3.61	181,000	4.01				280,000

TABLE 4

Unit Hydrograph Properties and Watershed Characteristics

Index No.	River	Station	Net Drainage Area Sq. Mi.	S Feet Per Sq. Mi.	No. of Stems	Peak of Distribution Graph %	Peak Discharge of Unit Hydrograph Q_{12}	Time Lag From Beginning of Rain to Peak of Run-off in Hours T_{R12}	Base of Unit Hydrograph in Days T_{T12}
1	Connecticut	North Stratford	716	-	1	10.25	7,810	18.0	6.50
2	Chicopee	Bircham Bend	703	2.05	3	9.9	7,400	36.0	6.50
3	White	West Hartford	687	3.21	3	21.6	15,900	18.0	4.25
4	Farmington	Tariffville	457	3.19	2	11.6	5,650	30.0	4.75
5	Westfield	Westfield	450	4.88	3	26.1	12,500	12.0	3.50
6	Passumpsic	Passumpsic	423	-	2	20.1	9,050	15.0	3.75
7	Ashuelot	Hinsdale	420	3.54	1	9.0	4,020	48.0	5.75
8	Millers	Erving	373	3.17	2	9.3	3,730	30.0	6.25
10	Ammonoosuc	Bath	393	6.75	2	22.9	9,520	15.0	4.25
11	West	Newfane	308	8.70	1	21.8	7,100	15.0	4.25
12	Sugar	West Claremont	232	8.12	1	16.8	4,150	18.0	4.50
13	White	Bethel	240	10.60	1	25.0	6,370	12.0	3.75
14	Ottauquechee	North Hartland	221	10.60	1-1/2	24.6	5,800	12.0	3.75
15	Farmington	Riverton	198	6.95	2	20.6	4,350	12.0	4.00
16	Ware	Gibbs Crossing	199	6.00	1-1/2	14.0	2,970	18.0	5.50
17	Swift	West Ware	186	4.53	3	10.8	2,140	36.0	6.50
18	Black	North Springfield	163	13.30	1	26.2	4,560	12.0	4.25
20	Mascoma	Mascoma	153	7.33	1	12.4	2,020	30.0	5.50
21	Quaboag	West Brimfield	151	4.97	1	7.8	1,240	36.0	5.75
23	Salmon	East Hampton	105	7.42	2	17.8	1,970	15.0	4.00
24	Scantic	Broad Brook	99	8.03	1	12.0	1,265	30.0	4.75
25	Farmington	New Boston	74	14.7	2	24.9	1,960	12.0	3.50

TABLE 5 - FLOOD ROUTING REACHES AND BASIC DATA - CONNECTICUT RIVER WATERSHED

REACH						X	T	K	C_0	C_1	C_2
	INFLOW										
No.	Tributary	Station	Drainage: Area	Miles above: Outflow Station	Outflow		(Days)				
1	Dalton	-	1,538	36.5	South Newbury	.3	.5	.88	-.02	.595	.425
	Passumpsic	Passumpsic	423	30.5	"	.3	.5	.72	.04	.615	.345
	Wells	Mouth	99	13.0	"	.3	.5	.58	.12	.645	.235
	Ammonoosuc	Bath	393	18.1	"	.3	.5	.61	.105	.635	.260
	Local #1	-	372	0.0	"	.3	.5	-	-	-	-
2	South Newbury	-	2,825	37.8	White R. Jct.	.3	.5	.67	.07	.63	.30
	Waits	Mouth	156	31.7	"	.3	.5	.38	.27	.71	.02
	Ompompanoosuc	Mouth	136	9.2	"	.3	.5	.04	-	-	-
	White	W. Hartford	690	7.5	"	.3	.5	-	-	-	-
	Local #2	-	261	0.0	"	.3	.5	-	-	-	-
3	White River Jct.	-	4,068	41.6	Bellows Falls	.3	.5	.53	.14	.66	.20
	Mascoma	Mascoma	153	50.6	"	.3	.5	.61	.105	.635	.260
	Ottauquechee	N. Hartland	221	38.0	"	.3	.5	.50	.165	.665	.170
	Sugar	W. Claremont	269	24.1	"	.3	.5	.42	.235	.693	.072
	Black	N. Springfield	158	17.7	"	.3	.5	.38	.27	.71	.02
	Local #3	-	518	0.0	"	.3	.5	-	-	-	-
4	Bellows Falls	-	5,387	31.7	Vernon	.3	.5	.48	.18	.67	.15
	Saxtons	Mouth	78	30.6	"	.3	.5	.48	.18	.67	.15
	West	Newfane	308	20.0	"	.3	.5	.26	.40	.76	-.16
	Local #4	-	466	0.0	"	.3	.5	-	-	-	-
5	Vernon	-	6,239	22.9	Montague City	.3	.5	.38	.27	.71	.02
	Ashuelot	Hinsdale	420	22.2	"	.3	.5	.38	.27	.71	.02
	Millers	Erving	370	14.8	"	.3	.5	.25	.41	.765	-.175
	Deerfield	Charlemont	362	25.3	"	.3	.5	.73	.04	.615	.345
	Local #5	-	449	0.0	"	.3	.5	-	-	-	-

TABLE 5 (CONT.) – FLOOD ROUTING REACHES AND BASIC DATA – CONNECTICUT RIVER WATERSHED

REACH						X	T (DAYS)	K	C_0	C_1	C_2
INFLOW											
No.	TRIBUTARY	STATION	DRAINAGE AREA	MILES ABOVE OUTFLOW STATION	OUTFLOW						
6	MONTAGUE CITY	–	7,840	51.1	THOMPSONVILLE	.3	.5	.64	.08	.63	.29
	CHICOPEE	BIRCHAM BEND	703	17.4	"	.3	.5	.36	.285	.715	.00
	WESTFIELD	WESTFIELD	497	14.8	"	.3	.5	.37	.28	.71	.01
	LOCAL #6	–	597	0.0	"	.3	.5	–	–	–	–
7	THOMPSONVILLE	–	9,637	16.0	HARTFORD	.3	.5	.99	–.05	.58	.47
	FARMINGTON	TARIFFVILLE	569	17.5	"	.3	.5	1.10	–.08	.57	.51
	SCANTIC	BROAD BROOK	99	13.2	"	.3	.5	.40	.245	.695	.06
	LOCAL #7	–	338	0.0	"	.3	.5	–	–	–	–
	HARTFORD		10,643								

TABLE 6
DETERMINATION OF "X"
CONNECTICUT RIVER FLOOD OF APRIL 16 - 25, 1933.
MONTAGUE CITY - THOMPSONVILLE REACH

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
	INFLOW							A	B	C	D				x = .0		x = .20		x = .30		x = .40	
Period .5 Day	Montague City* Thous.C.F.S.	Chicopee* Thous.C.F.S.	Westfield* Thous.C.F.S.	Local* Thous.C.F.S.	Inflow* Thous.C.F.S.	Adjusted* Inflow Thous.C.F.S.	Outflow at* Thompsonville Thous.C.F.S.	Thous.d.s.f. / 4	Thous.d.s.f. / 4	Thous.c.f.s.	Thous.c.f.s.	(A-B)/4	* W (A-B)/4	C-D	D+X(C-D)	* W[D+X(C-D)]	D+X(C-D)	* W[D+X(C-D)]	D+X(C-D)	* W[D+X(C-D)]	D+X(C-D)	* W[D+X(C-D)]
April 1933																						
15-1																						
2																						
16-1	60.0	4.7	5.2	4.0	73.9	72.5	70.5	151.6	146.5	6.6	5.5	1.3		1.1	5.5		5.7		5.8		5.9	
2	66.5	4.9	5.1	4.1	80.6	79.1	76.0	164.5	157.0	6.3	5.0	1.9	1.3	1.3	5.0	5.5	5.3	5.7	5.4	5.8	5.5	5.9
17-1	72.5	5.2	5.1	4.3	87.1	85.4	81.0	177.0	168.5	6.2	6.5	2.1	3.2	- .3	6.5	10.5	6.4	11.0	6.4	11.2	6.4	11.4
2	78.0	5.5	5.5	4.4	93.4	91.6	87.5	191.0	180.5	7.8	5.5	2.6	5.3	2.3	5.5	17.0	6.0	17.4	6.2	17.6	6.4	17.8
18-1	85.0	5.7	6.0	4.6	101.3	99.4	93.0	220.4	196.0	21.6	10.0	6.1	7.9	11.6	10.0	22.5	12.3	23.4	13.5	23.8	14.6	24.2
2	106.0	5.9	6.5	4.9	123.3	121.0	103.0	263.5	216.0	21.5	10.0	12.0	14.0	11.5	10.0	32.5	12.3	35.7	13.5	37.3	14.6	38.8
19-1	126.0	6.1	8.1	5.1	145.3	142.5	113.0	294.4	244.0	9.4	18.0	12.6	26.0	-8.6	18.0	42.5	16.3	48.0	15.4	50.8	14.6	53.4
2	135.5	6.4	7.6	5.3	154.8	151.9	131.0	301.8	285.0	- 2.0	23.0	4.2	38.6	-25.0	23.0	60.5	18.0	64.3	15.5	66.2	13.0	68.0
20-1	135.5	6.7	5.0	5.6	152.8	149.9	154.0	292.5	306.0	- 7.3	-2.0	-3.4	42.8	- 5.3	- 2.0	83.5	-3.1	82.3	-3.6	81.7	- 4.1	81.0
2	129.5	6.4	4.2	5.3	145.4	142.6	152.0	275.0	296.5	-10.2	-7.5	-5.4	39.4	- 2.7	- 7.5	81.5	-8.0	79.2	-8.3	78.1	- 8.6	76.9
21-1	120.5	6.0	3.5	5.0	135.0	132.4	144.5	255.6	278.5	- 9.2	-10.5	-5.7	34.0	+ 1.3	-10.5	74.0	-10.2	71.2	-10.1	69.8	-10.0	68.3
2	112.5	5.5	3.0	4.6	125.6	123.2	134.0	236.8	258.0	- 9.6	-10.0	-5.3	28.3	+ .4	-10.0	63.5	- 9.9	61.0	- 9.9	59.7	- 9.8	58.3
22-1	104.0	5.0	2.6	4.2	115.8	113.6	124.0	217.3	237.5	- 9.9	-10.5	-5.1	23.0	+ .6	-10.5	53.5	-10.4	51.1	-10.3	49.8	-10.3	48.5
2	95.0	4.5	2.4	3.8	105.7	103.7	113.5	197.5	216.5	- 9.9	-10.5	-4.7	17.9	+ .6	-10.5	43.0	-10.4	40.7	-10.3	39.5	-10.3	38.2
23-1	86.0	4.0	2.2	3.4	95.6	93.8	103.0	179.1	197.0	- 8.5	- 9.0	-4.5	13.2	+ .5	- 9.0	32.5	- 8.9	30.3	- 8.8	29.2	- 9.8	27.9
2	78.0	3.8	2.0	3.2	87.0	85.3	94.0	163.2	178.0	- 7.4	-10.0	-3.7	8.7	+ 2.6	-10.0	23.5	- 9.5	21.4	- 9.2	20.4	- 9.0	18.1
24-1	71.0	3.6	1.8	3.0	79.4	77.9	84.0	148.3	160.0	- 7.5	- 8.0	-2.9	5.0	+ .5	- 8.0	13.5	- 7.9	11.9	- 7.8	11.2	- 7.8	9.1
2	64.0	3.4	1.6	2.8	71.8	70.4	76.0	134.5	142.5	- 6.3	- 9.5	-2.0	2.1	+ 3.2	- 9.5	5.5	- 8.9	4.0	- 8.5	3.4	- 8.2	1.3
25-1	58.0	3.2	1.5	2.6	65.3	64.1	66.5						.1			-4.0		-4.9		-5.1		-6.9
	1783.5	96.5	78.9	80.2	2039.1	2000.3	2000.5															
					69.6	68.3	68.5		$\frac{1932.0}{1969.5} = .981$													
					1969.5	1932.0	1932.0															

*Rate at beginning of Period.

TABLE 7

ROUTING OF MARCH 1936 FLOOD THROUGH MONTAGUE CITY - THOMPSONVILLE COMPOSITE REACH

1	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6		
	Montague City to Thompsonville K = 0.67				X = 0.3	Chicopee to Thompsonville K = 0.36				X = 0.3	Westfield to Thompsonville K = 0.31				X = 0.3	Local at: Thompsonville	Total
Period: 1/2 day 3-36	in. (inflow) 1000 c.f.s.	C_o = .065	C_1 = .625	C_2 = .31	dn = Col. 4+5+6 1000 c.f.s.	inflow 1000 c.f.s.	C_o = .285	C_1 = .715	C_2 = 0	dn = Col. 4+5+6 1000 c.f.s.	in. (inflow) 1000 c.f.s.	C_o = .33	C_1 = .73	C_2 = -.06	dn = Col. 4+5+6 1000 c.f.s.	dn 1000 c.f.s.	discharge (Total) Thompsonville (2 dn)
		Approx. Equal					Approx.	Equal				Approx.	Equal				
17-1	65.0	4.3	40.5	20.2	65.0	6.5	1.9	4.6	0	6.5	4.2	1.5	3.1	-.3	4.2	3.4	79.1
-2	66.0	5.4	41.2	20.2	65.0	6.7	1.7	4.8	0	6.5	4.4	2.5	3.2	-.3	4.3	3.6	79.4
18-1	83.0	10.3	51.8	20.7	66.8	6.1	2.2	4.4	0	6.5	7.6	12.4	5.5	-.3	5.4	4.1	82.8
-2	159.0	15.1	99.0	25.6	82.8	7.6	3.6	5.4	0	6.6	37.6	13.2	27.5	-1.1	17.6	10.9	117.9
19-1	232.5	16.1	145.5	43.1	139.7	12.6	5.4	9.0	0	9.0	40.0	6.4	29.2	-2.4	39.6	23.0	211.3
-2	247.0	15.9	154.0	63.1	204.7	18.8	5.8	13.4	0	14.4	19.4	3.5	14.2	-2.0	33.2	24.0	276.3
20-1	245.5	15.2	153.0	72.1	233.0	20.4	5.5	14.6	0	19.2	10.5	2.4	7.7	-.9	15.7	19.0	286.9
-2	234.0	13.8	146.0	74.5	240.3	19.2	4.8	13.7	0	20.1	7.4	2.0	5.4	-.6	9.2	10.9	280.5
21-1	212.5	12.9	133.0	72.6	234.3	16.8	3.8	12.0	0	18.5	6.2	2.5	4.5	-.4	6.8	6.6	266.2
-2	198.5	12.2	124.0	67.8	218.5	13.3	3.3	9.5	0	15.8	7.6	3.1	5.5	-.4	6.6	6.1	247.0
22-1	188.5	11.5	117.5	63.1	204.0	11.6	3.5	8.3	0	12.8	9.5	1.9	6.9	-.5	8.2	10.2	235.2
-2	177.5	10.8	111.0	59.5	192.1	12.4	3.0	8.9	0	11.8	5.8	1.5	4.2	-.5	8.3	9.2	221.4
23-1	166.0	9.8	103.8	56.1	181.3	10.6	2.6	7.6	0	11.9	4.4	1.2	3.2	-.3	5.2	7.1	205.5
-2	151.0	8.6	94.0	52.5	169.7	9.2	2.3	6.6	0	10.2	3.6	1.1	2.6	-.3	4.1	5.2	189.2
24-1	132.0	7.2	82.1	48.1	155.1	8.2	2.1	5.9	0	8.9	3.2	.9	2.3	-.2	3.5	4.2	171.7
-2	111.5	6.3	69.5	42.6	137.4	7.4	1.7	5.3	0	8.0	2.9	1.0	2.1	-.2	3.0	3.9	152.3
25-1	96.5	5.7	60.4	36.8	118.4	6.1	1.6	4.4	0	7.0	3.1	1.3	2.3	-.2	2.9	3.8	132.1
-2	88.0	5.5	55.0	31.8	102.9	5.7	1.5	4.1	0	6.0	3.8	1.2	2.8	-.2	3.4	3.7	116.0
26-1	84.5		52.8		92.3	5.4		3.9	0	5.6	3.5		2.5	-.2	3.8	3.6	105.3
	2938.5				2903.3	204.6				205.3	184.7				185.0	162.5	3456.1

All values are rates at beginning of periods.

TABLE NO. 8

RELATIVE EFFICIENCIES OF COMPONENT AREAS CONTRIBUTING TO FLOOD CONTROL IN CONNECTICUT RIVER

Index Stations:		White River Junction		Bellows Falls		Vernon		Montague City		Thompsonville		Hartford	
Area:	Reaches	Reach #2		Reach #3		Reach #4		Reach #5		Reach #6		Reach #7	
No.	Drainage Area	D.A.= 4068 sq.mi.		D.A.= 5387 sq.mi.		D.A.= 6239 sq.mi.		D.A.= 7840 sq.mi.		D.A.= 9637 sq.mi.		D.A.= 10643 sq. mi.	
	sq. mi.	(M + N)/2	Cw.	(M + N)/2	Cw.	(M + N)/2	Cw.	(M + N)/2	Cw.	(M + N)/2	Cw.	(M + N)/2	Cw.
I	1538	105.8	39.5	94.4	27.0	91.7	22.0	65.6	13.1	44.3	7.0	46.1	6.7
II	1287	114.3	35.5	121.6	29.0	124.0	26.0	111.8	17.9	89.7	12.0	85.6	10.4
III	1243	67.7	20.4	96.0	22.3	111.0	22.2	143.9	23.0	146.5	19.0	138.5	16.2
IV	1319			88.6	21.7	90.8	19.0	121.3	20.6	140.8	19.3	134.9	16.7
V	852					54.5	7.6	84.4	9.3	123.7	10.9	124.9	10.0
VI	1601							93.1	18.6	98.5	16.4	120.0	18.1
VII	1797									85.7	15.9	96.0	16.2
VIII	1006											94.1	8.5
						STORM NO.2							
I	1538	98.3	36.3	68.0	19.5	48.0	11.5	35.2	7.0	31.2	5.0	25.7	3.7
II	1287	115.3	35.8	106.1	25.2	88.8	18.6	74.3	11.9	64.5	8.6	55.5	6.7
III	1243	74.8	22.4	118.5	27.5	130.8	26.2	130.4	20.9	123.5	16.1	112.7	13.2
IV	1319			97.7	23.8	116.5	24.5	124.4	21.1	124.6	17.1	117.0	14.6
V	852					88.2	12.4	108.6	11.9	123.8	11.0	126.4	10.2
VI	1601							101.7	20.3	111.2	18.5	116.0	17.5
VII	1797									84.0	15.5	141.0	23.8
VIII	1006											107.3	10.6
						STORM NO. 3							
I	1538	68.8	25.4	45.2	12.9	28.5	6.8	29.9	6.0	17.9	2.8	26.2	3.8
II	1287	107.3	33.3	87.7	20.8	64.8	13.6	64.9	10.4	45.2	6.0	57.3	7.0
III	1243	114.4	34.5	114.8	26.6	128.4	25.7	124.1	19.8	109.0	14.2	117.9	13.8
IV	1319			114.3	27.9	124.8	26.2	123.5	21.0	118.2	16.2	120.6	15.0
V	852					116.3	16.3	120.1	13.2	140.0	12.4	130.8	10.5
VI	1601							107.1	21.4	115.6	19.3	113.2	17.1
VII	1797									105.6	19.6	102.2	17.3
VIII	1006											96.4	8.8
						STORM NO. 4							
I	1538	97.0	35.7	69.3	19.8	50.9	12.2	36.8	7.4	31.1	5.0	25.7	3.7
II	1287	120.8	37.5	110.7	26.3	94.6	19.8	77.1	12.3	69.1	9.2	58.6	7.1
III	1243	84.5	25.4	124.9	29.0	141.5	28.3	140.3	22.5	133.7	17.4	118.3	13.8
IV	1319			104.2	25.4	125.6	26.4	134.5	22.9	135.4	18.6	128.9	16.0
V	852					95.2	13.2	116.8	12.9	136.4	12.1	141.2	11.3
VI	1601							110.0	22.0	120.7	20.1	123.2	18.6
VII	1797									94.6	17.5	112.8	19.1
VIII	1006											107.7	9.8

TABLE 9

EFFECT OF COMPREHENSIVE PLAN OF RESERVOIRS ON THE 1927, 1936, AND DEMONSTRATION FLOODS.

RIVER	STATION	NOVEMBER 1927 FLOOD						MARCH 1936 FLOOD						DEMONSTRATION FLOOD					
		NATURAL		MODIFIED		REDUCTION		NATURAL		MODIFIED		REDUCTION		NATURAL		MODIFIED		REDUCTION	
		STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)	STAGE (FT.)	DISCHARGE (C.F.S.)
PASSUMPSIC	PASSUMPSIC	30.9	25,200	22.6	17,300	8.3	7,900	21.2	16,000	16.1	11,200	5.1	4,800	30.4	24,700	22.8	17,500	7.6	7,200
WELLS	MOUTH	-	12,200	-	10,600	-	1,600	-	6,500	-	5,600	-	900	-	6,200	-	5,300	-	900
AMMONOOSUC	BATH	18.3	37,600	16.5	31,000	1.8	6,600	15.7	27,900	14.3	23,000	1.4	4,900	14.1	22,400	12.9	18,400	1.2	4,000
LOCAL #1	SOUTH NEWBURY	-	18,100	-	17,200	-	900	-	20,800	-	20,000	-	800	-	6,200	-	5,900	-	300
WAITS	MOUTH	-	21,100	-	16,300	-	4,800	-	7,900	-	6,400	-	1,500	-	10,000	-	8,000	-	2,000
OMPOMPANOOSUC	MOUTH	-	22,000	-	1,400	-	20,600	-	8,900	-	1,600	-	7,300	-	9,000	-	2,000	-	7,000
WHITE	WEST HARTFORD	29.3	70,300	20.9	43,300	8.4	27,000	18.9	37,000	15.4	25,700	3.5	11,300	19.9	40,000	15.2	25,100	4.7	14,900
OTTAUQUECHEE	NORTH HARTLAND	21.5	30,400	7.3	4,400	14.2	26,000	15.6	19,200	7.0	4,000	8.6	15,200	12.5	13,300	6.4	3,200	6.1	10,100
SUGAR	WEST CLAREMONT	8.8	9,400	2.8	800	6.0	8,600	10.9	14,000	5.5	3,600	5.4	10,400	11.2	14,600	6.0	4,400	5.2	10,200
BLACK	NORTH SPRINGFIELD	19.2	20,500	7.3	2,500	11.9	18,000	16.4	14,700	7.2	2,400	9.2	12,300	13.1	8,800	8.0	3,000	5.1	5,800
WEST	NEWFANE	23.0	53,100	7.2	2,600	15.8	50,500	19.3	39,000	8.0	4,000	11.3	35,000	13.2	17,900	8.0	3,900	5.2	14,000
ASHUELOT	HINSDALE	7.6	6,700	7.3	5,500	0.3	1,200	10.9	16,600	9.5	14,100	0.6	2,500	10.6	19,000	9.9	15,800	0.7	3,200
MILLERS	ERVING	5.8	5,600	4.6	2,800	1.2	2,800	10.9	19,700	8.4	12,600	2.5	7,100	9.8	16,500	7.5	10,100	2.3	6,400
WESTFIELD	WESTFIELD	25.4	42,500	20.3	27,500	5.1	15,000	27.2	48,200	23.4	36,200	3.8	12,000	20.7	28,700	18.1	22,000	2.6	6,700
CONNECTICUT	SOUTH NEWBURY	35.4	65,900	30.3	49,100	5.1	16,800	38.6	77,800	35.9	68,000	2.7	9,800	44.5	100,600	41.4	88,200	3.1	12,400
CONNECTICUT	WHITE RIVER JUNCTION	35.0	136,000	25.2	77,000	9.8	59,000	32.5	120,000	28.0	92,500	4.5	27,500	35.8	141,300	30.6	107,800	5.2	33,500
CONNECTICUT	BELLOWS FALLS	303.2	150,500	292.7	72,100	10.5	78,400	302.2	171,000	295.5	120,400	6.7	50,600	306.6	202,900	298.2	140,800	8.4	62,100
CONNECTICUT	VERNON (HEADWATER)	134.3	159,000	126.4	68,400	7.9	90,600	137.2	198,500	132.5	135,200	4.7	63,300	139.7	234,200	134.2	157,800	5.5	76,400
CONNECTICUT	VERNON (TAILWATER)	124.5	165,300	108.0	73,500	16.5	91,800	128.7	211,900	121.7	146,100	7.0	65,800	131.5	253,200	125.5	174,200	6.0	79,000
CONNECTICUT	MONTAGUE CITY	42.8	188,000	31.1	94,300	11.7	93,700	48.7	247,000	41.3	173,500	7.4	73,500	52.9	288,700	44.8	207,300	8.1	81,400
CONNECTICUT	HOLYOKE	14.8	188,000	9.2	94,300	5.6	93,700	16.8	247,000	14.1	173,500	2.7	73,500	17.9	288,700	15.5	207,300	2.4	81,400
CONNECTICUT	SPRINGFIELD	22.4	202,000	15.9	116,000	6.5	86,000	28.3	282,000	23.5	216,000	4.8	66,000	33.3	351,700	27.4	270,000	5.9	87,700
CONNECTICUT	THOMPSONVILLE	12.7	202,000	8.1	116,000	4.6	86,000	16.6	282,000	13.4	216,000	3.2	66,000	19.9	351,700	16.0	270,000	3.9	81,700
CONNECTICUT	HARTFORD	29.0	181,000	22.0	106,500	7.0	74,500	37.3	280,000	32.5	222,300	4.8	57,700	43.8	367,400	38.3	293,000	5.5	74,400

TABLE 10
RELATIVE EFFICIENCIES OF INDIVIDUAL TRIBUTARIES
IN FORMING 1936 FLOOD ON CONN. RIVER

INDEX STATIONS		SOUTH NEWBURY			WHITE RIVER JUNCTION			BELLOWS FALLS			VERNON			MONTAGUE CITY			THOMPSONVILLE			HARTFORD		
STREAM	D.A. SQMI	REACH #1			REACH #2			REACH #3			REACH #4			REACH #5			REACH #6			REACH #7		
		N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.
DALTON(CONN.R.)	1538																			84.1	35.8	60.0
PASSUMPSIC	423	125.0	111.5	118.2	132.6	104.5	118.6	135.5	95.0	115.2	97.6	65.5	81.6	108.0	66.2	87.1	106.0	56.3	86.2	105.0	78.5	91.8
WELLS	99	129.5	124.0	126.8	144.7	125.5	135.1	159.4	113.1	136.2	127.1	91.5	109.3	135.2	89.5	112.4	135.0	93.5	114.2	130.0	99.5	114.8
AMMONOOSUC	393	132.0	124.0	128.0	141.0	111.5	126.2	157.6	104.5	131.0	112.0	76.2	94.1	123.5	76.1	99.8	121.0	74.2	97.6	106.5	88.0	97.2
LOCAL #1	372	121.0	99.5	110.3	136.0	123.3	129.7	157.5	116.9	137.2	119.4	91.2	105.3	132.1	89.6	110.8	128.0	93.1	110.6	117.0	102.6	109.8
WAITS	156				134.5	157.5	146.0	153.4	136.5	145.0	137.0	116.0	126.5	138.1	116.5	127.3	134.0	119.0	126.5	128.2	125.7	127.0
OMPOMPANOOSUC	136				121.3	179.6	150.4	138.8	145.0	141.9	137.0	135.0	136.0	139.1	131.8	135.4	132.0	120.0	126.0	132.2	137.4	134.8
WHITE	690				129.0	212.0	170.5	144.5	121.0	132.8	131.0	120.5	125.8	131.5	118.1	124.8	125.5	117.0	121.2	113.5	131.5	122.5
LOCAL #2	261				133.4	203.0	168.2	152.0	166.5	159.2	142.2	134.0	138.1	141.0	130.0	135.5	135.0	130.2	132.6	130.0	142.3	136.2
MASCOMA	153							116.4	77.8	97.1	85.5	60.0	72.8	95.3	60.4	78.1	87.5	59.3	73.4	91.6	68.5	80.0
OTTAUQUECHEE	221							148.5	175.1	161.8	137.4	152.5	145.0	134.3	142.5	138.4	128.0	135.6	131.8	122.0	145.2	133.6
SUGAR	269							149.6	166.5	158.0	129.0	112.5	120.8	128.0	111.4	119.7	124.0	106.5	115.2	121.0	125.6	123.3
BLACK	158							158.0	198.7	178.4	138.0	198.0	168.0	137.0	131.8	134.4	132.0	125.0	128.5	127.0	141.7	134.3
LOCAL #3	518							152.5	145.0	148.8	158.6	192.3	172.4	146.2	181.0	163.6	139.5	173.6	156.6	132.0	169.9	151.0
SAXTONS	78										135.1	162.0	148.6	136.2	162.1	149.2	131.0	165.3	148.2	128.1	142.0	135.0
WEST	308										149.1	185.5	167.3	149.0	192.5	170.8	141.0	195.7	168.4	138.5	165.0	151.8
LOCAL #4	466										150.0	177.5	163.8	145.3	188.1	166.7	139.0	185.5	162.2	133.0	166.7	149.8
ASHUELOT	420													121.2	101.9	111.6	118.5	102.9	110.7	118.5	107.0	112.8
MILLERS	370													131.5	132.7	132.1	126.5	128.0	127.2	120.0	133.2	126.6
DEERFIELD	362													94.0	101.4	97.7	89.6	107.0	98.3	86.8	96.9	91.9
LOCAL #5	449													145.3	134.5	139.9	140.5	147.0	143.8	139.3	144.5	141.9
CHICOPEE	703																127.7	136.5	127.1	120.0	127.3	124.6
WESTFIELD	497																104.0	76.0	90.0	110.0	89.6	99.8
LOCAL #6	597																119.5	117.0	118.2	121.5	109.0	115.2
FARMINGTON	569																			141.0	147.0	105.0
SCANTIC	99																			73.1	67.1	70.1
LOCAL #7	297																			95.5	76.1	85.8

TABLE 11
RELATIVE EFFICIENCIES OF INDIVIDUAL
TRIBUTARIES IN FORMING DEMONSTRATION
FLOOD ON CONNECTICUT RIVER

INDEX STATIONS		SOUTH NEWBURY			WHITE RIVER JUNCTION			BELLOWS FALLS			VERNON			MONTAGUE CITY			THOMPSONVILLE			HARTFORD		
		REACH #1			REACH #2			REACH #3			REACH #4			REACH #5			REACH #6			REACH #7		
STREAM	D.A. SQMI	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.	N	M	AVE.
DALTON(CONN.R.)	1538																			63.5	41.0	52.3
PASSUMPSIC	423	127.0	149.1	138.0	139.0	113.2	128.6	120.0	84.2	102.1	99.0	63.6	81.3	87.4	60.2	73.8	78.0	52.1	65.0	80.5	56.0	68.2
WELLS	99	113.5	150.0	134.2	141.0	135.7	138.2	128.0	96.7	112.4	112.0	75.5	93.8	98.1	70.4	84.2	86.5	57.7	72.1	91.9	67.2	79.6
AMMONOOSUC	393	121.0	147.0	134.0	135.0	120.1	127.6	119.0	86.5	102.8	100.7	66.0	83.4	88.0	62.7	75.4	79.0	53.3	66.2	81.8	58.8	70.3
LOCAL #1	372	92.4	118.3	105.4	121.0	127.5	124.2	120.5	106.5	113.5	110.0	87.5	98.8	99.5	79.2	89.4	87.8	66.6	77.2	87.2	70.7	79.0
WAITS	156				134.0	157.7	145.9	144.0	150.8	147.4	142.4	138.5	140.4	137.0	125.2	131.1	124.1	111.6	117.8	117.0	111.0	114.0
OMPOMPANOOSUC	136				119.5	130.5	125.0	138.0	149.0	143.5	144.5	157.0	150.8	146.0	147.3	146.9	135.0	138.0	136.5	130.0	135.9	133.0
WHITE	690				128.0	141.8	134.9	138.4	143.0	140.7	138.0	140.0	139.0	135.4	129.8	132.6	123.2	115.4	119.3	119.0	116.3	117.7
LOCAL #2	261				128.0	145.8	136.9	142.0	149.0	145.5	144.0	147.2	145.6	143.0	136.5	139.8	130.4	122.9	126.4	124.0	122.5	123.2
MASCOMA	153							131.0	116.5	123.7	121.2	103.0	112.1	113.8	94.6	104.2	102.8	85.4	94.1	102.0	86.9	94.5
OTTAUQUECHEE	221							137.0	142.0	139.5	140.0	146.0	143.0	140.0	138.2	139.1	128.0	126.8	127.4	122.0	124.1	123.0
SUGAR	269							135.0	135.5	135.2	135.2	135.0	135.1	133.3	126.5	129.9	120.2	113.2	116.7	117.0	114.9	115.9
BLACK	158							132.8	131.3	132.1	136.0	140.0	138.0	137.0	133.9	135.4	127.5	125.6	126.6	121.0	122.4	121.7
LOCAL #3	518							122.0	123.3	122.6	134.0	147.0	140.5	142.0	146.9	144.4	137.5	145.5	141.5	129.0	143.9	136.4
SAXTONS	78										128.0	133.5	130.8	137.8	140.8	139.3	134.5	143.0	138.8	128.0	137.1	132.6
WEST	308										122.0	126.5	124.2	131.0	131.3	131.2	130.0	137.0	133.5	126.0	132.2	129.1
LOCAL #4	466										116.5	121.0	118.8	128.0	128.1	128.0	131.0	139.0	135.0	126.6	135.1	130.8
ASHUELOT	420													127.0	115.5	121.2	117.5	108.2	112.8	114.0	108.0	111.0
MILLERS	370													123.0	115.9	119.4	116.6	112.0	114.3	113.0	110.9	111.9
DEERFIELD	362													136.0	142.1	139.0	137.0	148.0	142.5	132.0	143.8	137.9
LOCAL #5	449													106.0	95.5	100.8	117.7	113.3	115.5	116.0	113.2	114.6
CHICOPEE	703																126.0	126.6	126.3	123.0	124.5	123.8
WESTFIELD	497																99.7	87.5	93.6	106.0	97.7	101.8
LOCAL #6	597																97.3	86.5	91.9	103.3	94.4	98.8
FARMINGTON	569																			123.3	123.9	123.6
SCANTIC	99																			104.0	99.3	101.6
LOCAL #7	297																			84.0	74.0	79.0

TABLE 12
AVERAGE RELATIVE EFFICIENCIES *
OF INDIVIDUAL TRIBUTARIES IN FORMING
FLOODS ON CONNECTICUT RIVER

INDEX STATIONS		SOUTH NEWBURY			WHITE RIVER JUNCTION			BELLOWS FALLS			VERNON			MONTAGUE CITY			THOMPSONVILLE			HARTFORD		
		REACH #1			REACH #2			REACH #3			REACH #4			REACH #5			REACH #6			REACH #7		
STREAM	D.A. SQ. MI.	(N + M)/2			(N + M)/2			(N + M)/2			(N + M)/2			(N + M)/2			(N + M)/2			(N + M)/2		
		D.F.•	1936	AVE.	D.F.•	1936	AVE.	D.F.•	1936	AVE.	D.F.•	1936	AVE.	D.F.•	1936	AVE.	D.F.•	1936	AVE.	D.F.•	1936	AVE.
DALTON (CONN.R.)	1538																			52.3	60.0	56.2
PASSUMPSIC	423	138.0	118.2	127.9	128.6	118.6	123.6	102.1	115.2	108.7	81.3	81.6	81.4	73.8	87.1	80.4	65.0	86.2	75.6	68.2	91.8	80.0
WELLS	99	134.2	126.8	130.5	138.2	135.1	136.6	112.4	136.2	124.3	93.8	109.3	101.6	84.2	112.4	98.3	72.1	114.2	93.1	79.6	114.8	97.2
AMMONOOSUC	393	134.0	128.0	131.0	127.6	126.2	126.9	102.8	131.0	116.9	83.4	94.1	88.8	74.5	99.8	87.2	66.2	97.6	81.9	70.3	97.2	82.7
LOCAL #1	372	105.4	110.3	107.8	124.2	129.7	127.0	113.5	137.2	125.4	98.8	105.3	102.0	89.4	110.8	100.1	77.2	110.6	93.9	79.0	109.8	94.4
WAITS	156				145.9	146.0	146.0	147.4	145.0	146.2	140.4	126.5	133.4	131.1	127.3	129.2	117.8	126.5	122.2	114.0	127.0	120.5
OMPOMPANOOSUC	136				125.0	150.4	137.7	143.5	141.9	142.7	150.8	136.0	143.4	146.9	135.4	141.2	136.5	126.0	131.2	133.0	134.8	133.9
WHITE	690				134.9	170.5	152.7	140.7	132.8	136.8	139.0	125.8	132.4	132.6	124.8	128.7	119.3	121.2	120.2	117.7	122.5	120.1
LOCAL #2	261				136.9	168.2	152.6	145.5	159.2	152.4	145.6	138.1	141.8	139.8	135.5	137.6	126.4	132.6	129.5	123.2	136.2	129.7
MASCOMA	153							123.7	97.1	110.4	112.1	72.8	92.4	104.2	78.1	91.2	94.1	73.4	83.8	94.5	80.0	87.2
OTTAUQUECHEE	221							139.5	161.8	150.6	143.0	145.0	144.0	139.1	138.4	138.7	127.4	131.8	129.6	123.0	133.6	128.3
SUGAR	269							135.2	158.0	146.6	135.1	120.8	128.0	129.9	119.7	124.8	116.7	115.2	115.9	115.9	123.3	119.6
BLACK	158							132.1	178.4	155.2	138.0	168.0	153.0	135.4	134.4	134.9	126.6	128.5	127.6	121.7	134.3	128.0
LOCAL #3	518							122.6	148.8	135.7	140.5	172.4	156.4	144.4	163.6	154.0	141.5	156.6	149.0	136.4	151.0	143.7
SAXTONS	78										130.8	148.6	139.7	139.3	149.2	144.3	138.8	148.2	143.5	132.6	135.0	133.8
WEST	308										124.2	167.3	145.8	131.2	170.8	151.0	133.5	168.4	151.0	129.1	151.8	140.4
LOCAL #4	466										118.8	163.8	141.3	128.0	166.7	147.4	135.0	162.2	148.6	130.8	149.8	140.3
ASHUELOT	420													121.2	111.6	116.4	112.8	110.7	111.8	111.0	112.8	114.6
MILLERS	370													119.4	132.1	125.8	114.3	127.2	120.8	111.9	126.6	119.2
DEERFIELD	362													139.0	97.7	118.4	142.5	98.8	120.4	137.9	91.9	113.7
LOCAL #5	449													100.8	139.9	120.4	115.5	143.8	129.6	114.6	141.9	128.2
CHICOPEE	703																126.3	127.1	126.7	123.8	124.6	124.2
WESTFIELD	497																93.6	90.0	91.8	101.8	99.8	100.8
LOCAL #6	597																91.9	118.2	105.0	98.8	115.2	107.0
FARMINGTON	569																			123.6	145.0	134.3
SCANTIC	99																			101.6	70.1	85.8
LOCAL #7	297																			79.0	85.8	82.4

• DEMONSTRATION FLOOD

* BASED ON AVERAGE OF VALUES FOR D.F. AND 1936 FLOODS.

TABLE 13

INDICES OF REDUCTIONS OF PEAK DISCHARGE AT CONNECTICUT RIVER INDEX STATIONS BY INDIVIDUAL RESERVOIRS

		INDEX STATIONS		SOUTH NEWBURY		WHITE RIVER JUNCTION		BELLOWS FALLS		VERNON		MONTAGUE CITY		THOMPSON-VILLE		HARTFORD	
		DAMAGE ZONES		ZONES 1&2		ZONE 3		ZONE 4		ZONE 5&6		ZONE 7		ZONE 8&9		ZONE 10	
		VALLEY STORAGE REACHES		REACH #1		REACH #2		REACH #3		REACH #4		REACH #5		REACH #6		REACH #7	
NO.	RESERVOIR	DRAINAGE AREA		D.A.=2825		D.A.= 4068		D.A.= 5387		D.A.=6239		D.A.=7840		D.A.=9637		D.A.=10,643	
		GROSS	NET	$\frac{M+N}{2}$	C_W	$\frac{M+N}{2}$	C_W	$\frac{M+N}{2}$	C_W	$\frac{M+N}{2}$	C_W	$\frac{M+N}{2}$	C_W	$\frac{M+N}{2}$	C_W	$\frac{M+N}{2}$	C_W
18A	East Haven	47.5		127.9	2.15	123.6	1.44	108.7	.97	81.4	.62	80.4	.49	75.6	.38	80.0	.36
21A	Lyndon Center	52		127.9	2.35	123.6	1.58	108.7	1.05	81.4	.68	80.4	.53	75.6	.41	80.0	.39
22A	Victory	66		127.9	2.98	123.6	2.00	108.7	1.33	81.4	.86	80.4	.68	75.6	.52	80.0	.50
50	Harvey Lake	24.9		107.8	.95	127.0	.78	125.4	.58	102.0	.41	100.1	.32	93.9	.24	94.4	.22
24A	Bethlehem Junction	90		131.0	4.17	126.9	2.80	116.9	1.95	88.8	1.28	87.2	1.00	81.9	.76	82.7	.71
26	Gale River	86		131.0	3.98	126.9	2.68	116.9	1.86	88.8	1.22	87.2	.96	81.9	.73	82.7	.67
69	Bath	397		131.0	18.40	126.9	12.38	116.9	8.60	88.8	5.65	87.2	4.42	81.9	3.38	82.7	3.10
27A	Groton Pond	17.3		130.5	.78	136.6	.57	124.3	.39	101.6	.28	98.3	.21	93.1	.16	97.2	.16
28A	South Branch	45		140.0	2.23	146.0	1.61	146.2	1.22	133.4	.96	129.2	.74	122.2	.57	120.5	.51
48	Union Village	126		132.0	5.88	137.7	4.25	142.7	3.33	143.4	2.90	141.2	2.28	131.2	1.72	133.9	1.59
29A	Gaysville	226				152.7	8.45	136.8	5.72	132.4	4.79	128.7	3.70	120.2	2.82	120.1	2.56
30A	Ayers Brook	30				152.7	1.13	136.8	.76	132.4	.64	128.7	.49	120.2	.37	120.1	.33
49A	South Tunbridge	102				152.7	3.82	136.8	2.59	132.4	2.17	128.7	1.67	120.2	1.27	120.1	1.16
70	Centerville	692				152.7	25.90	136.8	17.55	132.4	14.70	128.7	11.38	120.2	8.65	120.1	7.85
66	West Canaan	80				120.0	2.36	110.4	1.64	92.4	1.19	91.2	.93	83.8	.70	87.2	.66
72	Mascoma Lake *	153	73			120.0	2.15	110.4	1.50	92.4	1.09	91.2	.85	83.8	.64	87.2	.60
63	North Hartland	222				154.0	8.36	150.6	6.17	144.0	5.10	138.7	3.92	129.6	2.98	128.3	2.70
53A	Stocker Pond	35						146.6	.95	128.0	.72	124.8	.56	115.9	.42	119.6	.39
73	Spectacle Pond	65						146.6	1.76	128.0	1.34	124.8	1.04	115.9	.78	119.6	.73
64A	Claremont	245						146.6	6.65	128.0	5.04	124.8	3.92	115.9	2.95	119.6	2.77
36	Ludlow	56						155.2	1.62	153.0	1.38	134.9	.96	127.6	.74	128.0	.68
74	Perkinsville	142						155.2	4.10	153.0	3.50	134.9	2.44	127.6	1.87	128.0	1.72
55A	North Springfield	156						155.2	4.51	153.0	3.84	134.9	2.68	127.6	2.06	128.0	1.88
40A	Newfane	326								145.8	7.60	151.0	6.27	151.0	5.11	140.4	4.32
57A	Surry Mountain	100								120.0	1.92	116.4	1.49	111.8	1.16	114.6	1.08
59	Lower Naukeag	19.7										125.8	.32	120.8	.25	119.2	.22
60	Hydeville *	85	65.3									125.8	1.04	120.8	.82	119.2	.73
61A	Priest Pond	19										125.8	.31	120.8	.24	119.2	.21
65	Birch Hill *	176	156.3									125.8	2.50	120.8	1.96	119.2	1.76
62A	Tully	50										125.8	.80	120.8	.62	119.2	.56
47	Knightville	164												91.8	1.62	100.8	1.61

* – Indices are for net drainage areas
All values shown are percentages

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TABLE 14

SPILLWAY DATA AND GENERAL CHARACTERISTICS FOR CONNECTICUT RIVER FLOOD-CONTROL DAMS

Identification Number	Reservoir	River	Gross D.A.	F.C. Cap.	Type of Spillway	Type of Outlet	Elev. of Spillway Crest	Spillway-Design Floods: Summer and Fall: Vol.	Summer and Fall: Peak Discharge	Summer and Fall: Duration	Summer and Fall: Spillway Discharge	Summer and Fall: Spillway Surcharge	Winter and Spring: Rainfall Vol.	Rainfall Peak Discharge	Snow Vol.	Snow Peak Discharge	Rainfall & Snow Vol.	Rainfall & Snow Peak Discharge	Rainfall & Snow Duration	Spillway Dischg.	Spillway Surchg.	Governing Conditions	Coef. of Discharge	Spwy. Surcharge Storage	Fetch	Theoretical Freeboard Req'd	Freeboard Used	Distance Spillway Lip to Top of Dam
			Sq.Mi.	In.			Feet	In.	c.f.s.	Days	c.f.s.	Feet	In.	c.f.s.	In./day	c.f.s.	In.	c.f.s.	Days	c.f.s.	Ft.			In.	Miles	Feet	Feet	Feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)
18A	East Haven	Passumpsic	47.5	6.1	Saddle	Retarding	1040.	14.1	21,800	3.00	11,730	10	6.9	10,700	1.8	2,300	12.3	13,000	3.00			Summer	3.4	2.0	3.4	4.7	5.0	15.0
21A	Lyndon Center	Millers Run	52.	6.0	Saddle	"	766.5	14.0	20,800	3.75	8220	8	6.9	10,200	1.8	2,500	13.6	12,700	3.75			"	3.5	1.6	3.2	4.6	5.0	13.0
22A	Victory	Moose	66.	7.0	Side-Hill	"	1149.	14.	25,500	4.00	2200	10	8.8	15,600	1.8	3,170	16.0	18,800	4.00	3,350	10	Winter	3.4	5.6	3.0	4.5	5.0	15.0
50	Harvey Lake	Stevens	24.9	5.9	Saddle	"	900.	15.7	12,400	3.00	8,360	6	7.1	5,600	1.8	1,200	12.5	6,800	3.00			Summer	3.3	2.0	1.5	4.2	5.0	11.0
24A	Bethlehem Jct.	Ammonoosuc	90.	6.0	Side-Ch'l	"	1356.	14.6	48,100	3.25	20,200	12	10.1	33,200	1.8	4,300	16.0	37,500	3.25			"	3.2	2.8	3.2	4.6	5.0	17.0
26	Gale River	Gale River	86.	2.9	" "	"	912.	14.7	51,400	3.75	38,000	10	9.2	32,200	1.8	4,130	16.0	36,300	3.75			"	3.6	1.0	1.4	4.2	5.0	15.0
69	Bath	Ammonoosuc	397.	6.0	" "	"	600.	13.0	103,500	5.00	50,750	12	8.1	64,500	1.8	19,050	17.1	83,500	5.00			"	3.6	1.5	2.1	4.4	5.0	17.0
27A	Groton Pond	Wells	17.3	7.0	Overflow	"	1085.	15.9	8540	3.00	4,500	4	7.2	3,870	1.8	820	12.6	4700	3.00			"	3.6	2.3	2.5	4.4	5.0	9.0
28A	South Branch	S. Branch (Waits)	45.	6.0	Morn Glory	"	810.	15.3	21,900	3.00	12,080	10	6.9	9,860	1.8	2,160	12.3	12,000	3.00			"	3.6	1.3	0.6	3.8	5.0	15.0
48	Union Village	Ompompanoosuc	126.	4.5	" "	Gates	543.	13.7	52,900	4.00	48,200	13	7.4	28,400	2.1	7,060	15.8	35,500	4.00			"	3.5	1.2	2.3	4.4	5.0	18.0
29A	Gaysville	White	226.	6.5	Overflow	"	795.	14.2	64,800	5.00	57,100	12.75	7.1	32,500	2.1	12,680	17.6	45,200	5.00			"	3.6	2.0	1.8	4.3	0.0	10.0
30A	Ayers Brook	Ayers Brook	30.	6.0	Side Ch'l	Retarding	695.	16.2	16,150	3.25	6,710	7	7.0	6,960	2.1	1,680	13.8	8600	3.25			"	3.5	2.6	2.4	4.4	5.0	12.0
49A	S. Tunbridge	First Branch	102.	4.5	" "	Gates	553.	14.5	48,900	4.00	43,600	10	7.5	25,200	2.1	5,710	15.9	30,900	4.00			"	3.6	1.5	1.4	4.2	5.0	15.0
70	Centerville	White	692.	4.2	O.G.	Semi-Ret.	508.	12.2	201,000	4.50	75,200	16	6.4	105,100	2.1	38,700	15.8	143,800	4.50			"	3.6	1.6	2.9	4.5	5.0	21.0
66	West Canaan	Mascoma	80.	6.0	O.G.	Gates	893.	14.2	25,900	4.00	18,600	8	7.7	14,100	2.1	4,480	16.1	18,600	4.00			"	3.6	3.6	2.4	4.4	0.0	8.0
72	Mascoma Lake	Mascoma	153.	2.1	O.G.	"	759.	13.5	36,600	5.25	28,900	12	7.3	19,800	2.1	8,570	18.3	28,400	5.25			"	3.4	2.7	1.2	4.1	5.0	17.0
63	N. Hartland	Ottauquechee	222.	4.1	Side-Ch'l	"	528.	13.1	64,500	4.50	63,320	10	7.1	35,000	2.1	12,380	16.6	47,400	4.50			"	3.5	0.4	2.7	4.5	5.0	15.0
53A	Stocker Pond	Stocker Brook	35.	6.0	Side-Hill	Retarding	1032.	14.9	14,560	3.25	3,275	10	8.0	7,800	2.1	1,960	14.8	9,800	3.25			"	3.5	5.8	0.9	4.0	5.0	15.0
64A	Claremont	Sugar	245.	4.6	Side-Ch'l	Gates	607.	13.0	56,100	5.00	54,700	10	7.0	30,200	2.1	13,710	17.5	43,900	5.00			"	3.6	1.1	2.9	4.5	5.0	15.0
36	Ludlow	Black	56.	4.5	" "	"	1057.	15.1	30,400	3.50	23,350	10	8.9	17,900	2.1	3,140	16.2	21,000	3.50			"	3.6	2.8	2.3	4.4	5.0	15.0
74	Perkinsville	Black	142.	6.0	Overflow	"	635.	13.6	60,160	4.00	54,300	8	8.8	38,900	2.1	7,950	17.2	46,850	4.00			"	3.6	1.5	1.7	4.3	5.0	13.0
55A	N. Springfield	Black	156.	3.2	Side-Hill	"	519.	13.5	53,200	4.25	51,950	8.5	8.2	32,400	2.1	8,740	17.1	41,100	4.25			"	3.6	0.8	3.1	4.6	5.0	13.5
40A	Newfane	West	326.	6.0	Side-Ch'l	"	486.	13.2	85,900	5.00	78,800	10	7.8	50,700	2.1	18,250	18.3	68,900	5.00			"	3.6	1.3	1.9	4.4	5.0	15.0
57A	Surry Mountain	Ashuelot	100.	6.0	" "	"	541.	13.4	47,500	4.00	37,300	10	7.5	26,500	2.1	5,600	15.9	32,100	4.00			"	3.5	2.2	2.6	4.5	5.0	15.0
59	Lower Naukeag	Millers	19.7	5.1	O.G.	"	1076.	14.0	7900	3.25	4,800	4	9.8	5,500	2.1	1,120	16.6	6600	3.25			"	3.4	3.2	1.2	4.1	4.0	8.0
60	Hydeville	Millers	85.	3.2	O.G.	"	875.	13.1	21,800	4.25	19,700	10	9.2	15,300	2.1	4,760	18.1	20,100	4.25			"	3.5	2.1	2.1	4.4	5.0	15.0
61A	Priest Pond	Priest Brook	19.	6.0	Side-Hill	"	879.	14.0	8270	3.50	4,000	9	9.8	5,770	2.1	1,060	17.2	6800	3.50	5,070	9	Winter	3.4	5.2	2.8	4.5	5.0	14.0
65	Birch Hill	Millers	176.	5.3	Side-Hill	"	847.	12.4	32,100	6.00	22,600	12	8.7	22,600	2.1	9,860	21.3	32,500	6.00	26,000	12	"	3.5	2.5	2.6	4.5	5.0	17.0
62A	Tully	Tully	50.	8.0	Saddle	"	668.	13.6	20,700	3.50	12,370	10	9.5	14,500	2.1	2,800	16.8	17,300	3.50			Summer	3.6	4.7	1.4	4.2	5.0	15.0
47	Knightville	Westfield	164.	4.5	Saddle	"	596.	12.9	51,900	4.50	48,900	10	8.7	34,900	2.1	9,200	18.2	44,100	4.50			Summer	4.5	1.0	0.8	3.9	5.0	15.0

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TABLE 15

OUTLET DATA AND GENERAL CHARACTERISTICS FOR CONNECTICUT RIVER FLOOD CONTROL DAMS

IDENTIFICATION NO.	RESERVOIR	RIVER	GROSS DRAINAGE AREA SQUARE MILES	CAPACITY RUN OFF IN INCHES	TYPE RETARDING OR GATES	NUMBER AND TYPE OF CONDUIT USED	ELEVATIONS FEET M.S.L. Spillway Crest	ELEVATIONS FEET M.S.L. Top of Outlet	ELEVATIONS FEET M.S.L. Outlet Invert	MAXIMUM OPERATING HEAD IN FEET	CONDUIT LENGTH IN FEET	K_i in % of $\frac{V^2}{2g}$	K_f in % of $\frac{V^2}{2g}$	VELOCITY IN FEET PER SECOND	MAXIMUM NORMAL RETARDING BASIN DISCHARGE IN C.F.S.	FACTOR OF FLEXIBILITY FOR GATE OPERATION	OUTLET DESIGN DISCHARGE IN C.F.S.	C.F.S. PER SQUARE MILE	CONDUIT AREA RECOMMENDED SQUARE FEET	CONDUIT AREA USED SQUARE FEET	TYPE OF GATES	NUMBER AND SIZE OF GATES
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
18A	East Haven	Passumpsic	47.5	6.1	R	Horseshoe	1040.0	962.3	953.0	77.7	610.0	0.10	1.35	44.9	1,900	1.0	1,900	40	42.0	72.0	None	None
21A	Lyndon Center	Millers Run	52.0	6.0	R	Horseshoe	766.5	708.0	701.0	58.5	410.0	0.10	0.80	44.0	2,300	1.0	2,300	44	52.0	50.0	None	None
22A	Victory	Moose	66.0	7.0	R	Horseshoe	1149.0	1126.0	1119.5	23.0	220.0	0.10	0.39	31.5	1,850	1.0	1,850	28	59.0	58.0	None	None
50	Harvey Lake	Stevens	24.9	5.9	R	Horseshoe	900.0	887.5	880.0	12.5	150.0	0.10	0.25	24.3	1,550	1.0	1,550	62	64.0	63.0	None	None
24A	Bethlehem Junction	Ammonoosuc	90.0	6.0	R	Horseshoe	1356.0	1223.5	1212.5	132.5	1310.0	0.10	2.10	50.8	3,200	1.0	3,200	36	63.0	100.0	None	None
26	Gale River	Gale	86.0	2.9	R	Horseshoe	912.0	850.2	837.0	61.8	530.0	0.10	0.53	49.0	6,800	1.0	6,800	79	138.0	144.0	None	None
69	Bath	Ammonoosuc	397.0	6.0	R	Horseshoe	600.0	447.0	458.0	123.0	930.0	0.10	0.90	62.8	9,000	1.0	9,000	23	143.0	303.0	None	None
27A	Groton Pond	Wells	17.3	7.0	R	6 Circular	1085.0	1074.5	1072.0	12.25	12.5	0.10	0.13	25.4	660	1.0	660	38	26.0	30.0	None	None
28A	South Branch	South Branch	45.0	6.0	R	Horseshoe	810.0	738.0	730.0	72.0	210.0	0.10	0.51	53.4	1,900	1.0	1,900	42	36.0	72.0	None	None
48	Union Village	Ompompanoosuc	126.0	4.5	G	Horseshoe	543.0	432.0	417.0	111.0	450.0	0.15	0.43	67.0	8,400	1.2	10,100	79	151.0	152.0	Broome	2 - 7.5'x10.0'
29A	Gaysville	White	226.0	6.5	G	4 Circular	795.0	647.5	642.5	150.0	55.0	0.15	0.06	89.0	6,000	1.2	7,200	32	81.0	82.0	Butterfly	4 - 61" dia.
30A	Ayers Brook	Ayers Brook	30.0	6.0	R	Horseshoe	695.0	650.5	645.0	44.5	330.0	0.10	0.79	39.2	1,400	1.0	1,400	48	36.0	37.0	None	None
49A	South Tunbridge	First Branch	102.0	4.5	G	Horseshoe	553.0	501.0	489.0	52.0	440.0	0.15	0.39	46.0	6,200	1.2	7,440	73	162.0	167.0	Cast-steel	3 - 6.7'x10.0'
70	Centerville	White	692.0	4.2	G	8 Rectangular	508.0	391.0	381.0	127.0	64.0	0.15	0.03	83.2	27,000	1.3	35,100	51	422.0	454.0	Broome	6 - 5.8'x10.0'
66	West Canaan	Mascoma	80.0	6.0	G	2 Rectangular	893.0	878.0	858.0	25.0	50.0	0.15	0.07	35.6	2,600	1.2	3,120	39	88.0	96.0	Cast-steel	2 - 6' x 8'
72	Mascoma Lake	Mascoma	153.0	2.1	G	4 Rectangular	759.0	747.0	737.0	12.0	40.0	0.15	0.08	25.0	1,040	1.2	1,250	8	50.0	192.0	Cast-steel	4 - 6' x 8'
63	North Hartland	Ottauquechee	222.0	4.1	G	Horseshoe	528.0	410.5	394.0	117.5	730.0	0.15	0.53	66.7	11,200	1.3	14,550	66	218.0	226.0	Broome	3 - 7.5'x12.0'
53A	Stocker Pond	Stocker Brook	35.0	6.0	R	Horseshoe	1032.0	1004.6	997.5	27.4	230.0	0.10	0.50	33.1	1,400	1.0	1,400	40	42.0	42.0	None	None
64A	Claremont	Sugar	245.0	4.6	G	Horseshoe	607.0	536.0	522.0	71.0	475.0	0.15	0.33	55.6	9,300	1.3	12,100	49	220.0	238.0	Broome	3 - 8.0'x12.0'
36	Ludlow	Black	56.0	4.5	G	Horseshoe	1057.0	1006.5	999.0	50.5	290.0	0.15	0.36	46.0	3,500	1.3	4,500	80	98.0	98.0	Cast-steel	3 - 6.0'x8.0'
74	Perkinsville	Black	142.0	6.0	G	Horseshoe	635.0	552.0	540.0	58.0	450.0	0.15	0.50	47.2	4,200	1.4	5,880	41	124.0	119.0	Broome	3 - 6.0'x8.0'
55A	North Springfield	Black	156.0	3.2	G	3 Semi-Circ.	519.0	461.0	451.0	63.0	41.0	0.15	0.03	59.0	9,400	1.4	13,150	84	222.0	225.0	Cast-steel	3 - 7.5'x10.0'
40A	Newfane	West	326.0	6.0	G	Horseshoe	486.0	383.4	368.0	102.6	1020.0	0.15	0.76	58.2	8,000	1.5	12,000	37	206.0	212.0	Broome	3 - 8.25'x10.0'
57A	Surry Mountain	Ashuelot	100.0	6.0	G	Horseshoe	541.0	495.9	484.0	45.1	480.0	0.15	0.54	41.8	3,900	1.2	4,700	47	112.0	120.0	Broome	2 - 7.0'x10.0'
59	Lower Naukeag	Millers	19.7	5.1	G	2 Rectangular	1076.0	1064.0	1057.0	15.5	25.0	0.15	0.05	28.7	940	1.4	1,315	67	46.0	49.0	Cast-steel	2 - 3.5'x7.0'
60	Hydeville	Millers	85.0	3.2	G	Open-channel	875.0	875.0	840.0	35.0	-	-	-	-	5,500	1.5	8,250	97	10' wide	10' wide	Stoney	10'x35'
61A	Priest Pond	Priest Pond Brook	19.0	6.0	G	Open-channel	879.0	879.0	863.0	16.0	-	-	-	-	780	1.3	1,010	53	5.5' wide	5.5' wide	Tainter	5.5'x16.0'
65	Birch Hill	Millers	176.0	5.3	G	Open-channel	847.0	847.0	820.0	27.0	-	-	-	-	5,000	1.7	8,500	55	22' wide	22' wide	Tainter	2 - 11.0' x 27.0'
62A	Tully	Tully	50.0	8.0	G	Horseshoe	668.0	628.0	620.0	40.0	320.0	0.15	1.07	35.5	600	1.3	780	16	22.0	50.0	Cast-steel	2 - 4.0'x7.5'
47	Knightville	Westfield	164.0	4.5	G	Horseshoe	596.0	496.0	480.0	100.0	650.0	0.15	.48	62.8	8,000	1.7	13,600	83	216.0	216.0	Broome	3 - 7.0'x12.0'

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TABLE 31— REDUCTION OF FLOOD LOSSES BY COMPREHENSIVE PLAN OF RESERVOIRS

RIVER	ZONE NO.	INDEX STATION	1927 RECURRING DIRECT FLOOD LOSSES: NATURAL	1927 RECURRING DIRECT FLOOD LOSSES: REDUCTION BY COMPR. RES. PLAN	1936 RECURRING DIRECT FLOOD LOSSES: NATURAL	1936 RECURRING DIRECT FLOOD LOSSES: REDUCTION BY COMPR. RES. PLAN	AVERAGE ANNUAL FLOOD LOSSES — NATURAL: DIRECT	NATURAL: INDIRECT	NATURAL: DEPRECIATION OF PROPERTY VALUE*	NATURAL: TOTAL	REDUCTION BY COMP. RESERVOIR PLAN: DIRECT	REDUCTION: INDIRECT	REDUCTION: RESTORATION OF PROPERTY VALUE	REDUCTION: TOTAL
Connecticut	C-1	U.S.G.S. gage at South Newbury, Vt.	$ 30,000	$ 21,000	$ 104,600	$ 67,000	$ 16,500	$ 15,610	$ 2,590	$ 34,720	$ 5,750	$ 5,430	$ 2,590	$ 13,770
"	C-2	do.	423,000	323,000	647,800	221,000	89,860	84,920	2,550	177,330	39,030	36,880	2,550	78,460
"	C-3	" " at White River Junction, Vt.	640,000	640,000	310,200	247,000	34,280	32,390	11,040	77,710	28,780	27,200	11,040	67,020
"	C-4	New England Power Association Dam, Bellows Falls, Vt.	590,000	578,000	308,200	272,000	21,540	20,360	8,690	50,590	17,180	16,240	8,690	42,110
"	C-5	Headwater New England Power Assn. Dam, Vernon, Vt.	580,000	480,000	1,004,800	600,000	108,560	102,590	25,920	237,070	79,320	74,960	25,920	180,200
"	C-6	Tailwater New England Power Assn. Dam, Vernon, Vt.	244,000	244,000	617,300	502,000	13,740	12,980	3,890	30,610	11,980	11,320	3,890	27,190
"	C-7	U.S.G.S. gage at Montague City, Mass.	500,000	500,000	5,445,900	5,170,000	69,540	65,720	94,510	229,770	62,350	58,920	94,510	215,780
"	C-8	Memorial Bridge at Springfield, Mass.	600,000	570,000	8,157,800	5,030,000	134,000	126,630	79,540	340,170	107,500	101,590	79,540	288,630
"	C-9	U.S.G.S. gage at Thompsonville, Conn.	128,000	119,000	324,400	175,000	15,130	14,300	21,750	51,180	11,320	10,700	21,750	43,770
"	C-10	Memorial Bridge at Hartford, Conn.	1,050,000	1,000,000	10,677,000	5,150,000	204,500	193,250	125,280	523,030	164,640	155,580	125,280	445,500
		Main River Totals	4,785,000	4,475,000	27,598,000	17,434,000	707,670	668,750	375,760	1,752,180	527,850	498,820	375,760	1,402,430
Passumpsic	Tr-1f	Highway Bridge at Lyndon, Vt. (18.5 miles above mouth)	605,000	585,000	22,100	17,000	30,750	29,060	6,050	65,860	18,910	17,870	6,050	42,830
"	-1d	Highway Bridge at St.Johnsbury,Vt.(0.2mile above mouth of Moose R.)	64,000	60,500	4,500	3,500	2,990	2,830	820	6,640	2,430	2,300	820	5,550
"	-1e	U.S.G.S. gage at Passumpsic, Vt.	590,000	470,000	40,500	40,500	37,080	35,040	1,060	73,180	30,220	28,560	1,060	59,840
Stevens	-2	Green Mountain Power Corp. Dam (0.6 miles above mouth)	6,200	4,000	3,000	1,800	690	650	50	1,390	520	490	50	1,060
Wells	-3	Railroad Bridge (6.2 miles above mouth)	225,000	145,000	4,800	3,000	10,840	10,240	310	21,390	5,500	5,200	310	11,010
Ammonoosuc	Tr-4a	Littleton Water & Light Co. Dam (24.9 miles above mouth)	115,000	113,800	28,900	23,000	7,340	6,940	580	14,860	6,460	6,100	580	13,140
"	-4b	U.S.G.S. gage at Bath, N.H.	337,000	227,000	34,400	14,000	16,120	15,230	480	31,830	8,540	8,070	480	17,090
Waits	-5	Central Vermont Public Service Co. Dam (1.0 miles above mouth)	3,600	2,900	200	100	180	170	120	470	130	120	120	370
White	-7a	Locust Creek Dam Site (26.8 miles above mouth)	264,000	264,000	8,000	8,000	10,480	9,900	1,440	21,820	10,480	9,900	1,440	21,820
"	-7e	Railroad Bridge (0.7 miles above mouth of Third Branch)	205,000	110,000	0	0	4,900	4,630	1,150	10,680	2,050	1,940	1,150	5,140
"	Tr-7b	Highway Bridge at North Royalton, Vt. (21.7 miles above mouth)	127,500	115,300	4,300	3,500	6,380	6,030	480	12,890	5,120	4,840	480	10,440
"	-7c	U.S.G.S gage at West Hartland, Vt.	318,000	263,000	29,300	28,000	17,480	16,520	1,010	35,010	14,050	13,280	1,010	28,340
Mascoma	-8a	Highway Bridge (18.2 miles above mouth)	300	0	18,200	0	1,900	1,800	0	3,700	0	0	0	0
"	-8b	American Woolen Co. Dam, Lebanon, N.H. (4.7 miles above mouth)	21,500	0	42,500	0	14,420	13,630	0	28,050	0	0	0	0
Ottauquechee	-9	A. Dewey Co. Dam, Quechee, Vt. (5.7 miles above mouth)	60,000	0	12,100	0	4,030	3,810	0	7,840	0	0	0	0
Sugar	Tr-10a	Railroad Bridge (12.2 miles above mouth)	29,500	0	37,800	0	7,310	6,910	0	14,220	0	0	0	0
"	-10b	U.S.G.S. gage at Claremont, N.H.	20,500	20,500	51,400	51,400	7,620	7,200	4,320	19,140	6,580	6,220	4,320	17,120
Black	-11a	Vermont Hydro-Electric Co. Dam, Perkinsville, Vt. (11.9 miles above mouth)	168,000	0	26,600	0	10,130	9,570	0	19,700	0	0	0	0
"	-11b	U.S.G.S. gage at North Springfield, Vt.	295,000	295,000	40,500	40,500	17,350	16,400	960	34,710	13,360	12,360	960	26,950
Saxons	-12	Thompson & Thompson Co. Dam (5.4 miles above mouth)	7,000	0	3,300	0	430	410	0	840	0	0	0	0
West	Tr-13	Highway Bridge—West Dummerston, Vt. (5.0 miles above mouth)	335,000	335,000	31,600	31,600	9,040	8,540	1,340	18,920	9,040	8,540	1,340	18,920
"	Tr-13x	Highway Bridge, Jamaica, Vt. (26.0 miles above mouth)	196,000	0	78,800	0	10,400	9,830	0	20,230	0	0	0	0
Ashuelot	-14g	" " Westport, N.H. (16.6 miles above mouth)	25,000	20,000	246,600	79,000	26,780	25,310	25,680	77,770	14,800	13,990	25,680	54,470
"	-14f	Public Service Co. of N.H. Dam Tailwater (4.0 miles above mouth)	9,000	3,000	193,600	75,000	18,200	17,200	2,880	38,280	8,120	7,670	2,880	18,670
Millers	-15c	(31.4 miles above mouth)	0	0	650,100	80,000	48,500	45,830	9,600	103,930	16,850	15,920	9,600	42,370
"	Tr-15e	Railroad Bridge, South Royalston, Mass. (23.4 miles above mouth)	5,000	5,000	521,400	460,000	26,330	24,880	20,160	71,370	24,080	22,760	20,160	67,000
"	-15f	Pequoig Ave. Bridge, Athol, Mass. (1.4 miles above mouth of Tully River)	2,500	2,500	27,000	12,000	2,420	2,290	0	4,710	1,810	1,710	0	3,520
"	-15g	Chase Turbine Co. Dam Tailwater, Orange, Mass. (13.3 miles above mouth)	10,000	10,000	1,073,200	710,000	60,620	57,290	35,040	152,950	50,700	47,910	35,040	133,650
"	-15h	Railroad Bridge (7.0 miles above mouth)	0	0	138,200	112,000	4,400	4,160	8,160	16,720	3,580	3,380	8,160	15,120
Deerfield	-16a	Outlet of Harriman Reservoir	177,000	0	132,700	0	6,460	6,100	0	12,560	0	0	0	0
"	Tr-16b	U.S.G.S. gage at Charlemont, Mass.	357,000	0	272,600	0	23,600	22,300	0	45,900	0	0	0	0
Swift	-17a	U.S.G.S. " " West Ware, Mass.	0	0	41,100	0	2,970	2,810	0	5,780	0	0	0	0
Chicopee	-17b	U.S.G.S. " " Bircham Bend, Mass.	5,000	0	70,100	0	8,900	8,410	0	17,310	0	0	0	0
Westfield	-18	U.S.G.S. " " Westfield, Mass.	160,000	143,000	313,300	195,000	19,320	18,260	28,800	66,380	10,140	9,580	28,800	48,520
Farmington	-19a	U.S.G.S " " New Boston, Mass.	2,200	0	1,600	0	180	170	0	350	0	0	0	0
"	Tr-19b	U.S.G.S. gage at Riverton, Conn.	118,200	0	118,200	0	9,240	8,730	0	17,970	0	0	0	0
"	-19c	Highway Bridge Avon, Conn. (22.0 miles above mouth)	66,000	0	33,200	0	2,440	2,310	0	4,750	0	0	0	0
Quaboag	-21	Central Mass. Electric Co. Dam (5.1 miles above mouth)	0	0	96,500	0	10,200	9,640	0	19,840	0	0	0	0
Ware	-22	U.S.G.S. gage at Gibbs Crossing, Mass.	0	0	285,700	0	11,400	10,770	0	22,170	0	0	0	0
		Tributary Totals	4,930,000	3,194,500	4,737,900	1,988,900	509,820	481,800	150,490	1,142,110	263,470	248,980	150,490	662,940
		Grand Total	9,715,000	7,669,500	32,335,900	19,422,900	1,217,490	1,150,550	526,250	2,894,290	791,320	747,800	526,250	2,065,370

* Not including depreciation of property values in urban areas to be protected by dikes.

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TABLE 32

COMPUTATION OF BENEFITS TO LYNDON CENTER FROM REDUCTION OF DIRECT FLOOD LOSS

TRIBUTARY BENEFITS

Damage Zone	Frequency Range	Drainage Areas: Dam Site sq.mi	Drainage Areas: Index Station sq. mi.	L Ratio	C_T Ratio	R Ratio	Total Benefit Dollars	Benefit to Victory Dollars	Benefit to Lyndon Center Dollars
1f	A	54	210	.624	.123	.9	4,680	0	4,680
1f	B	54	210	.884	.176	.9	5,360	0	5,360
1f	C	54	210	1.000	.196	.9	4,040	0	4,040
1e	A	54	423	.631	.150*	1.0	10,950	6,800	4,150
1e	B	54	423	.887	.205*	1.0	8,250	5,200	3,050
1e	C	54	423	1.000	.220*	1.0	3,020	2,700	320
					Totals		36,300	14,700	21,600

*Includes effect of Victory Reservoir.

MAIN RIVER BENEFITS

Damage Zone	Frequency Range	C_W %	L Ratio	R Ratio	C_W L R %	U Dollars per%	Benefit Dollars
1	A	2.35	.897	1.0	2.108	403	850
1	B	2.35	1.0	1.0	2.350	179	420
2	A	2.35	.780	1.0	1.833	180	330
2	B	2.35	.981	1.0	2.305	530	1,220
2	C	2.35	1.0	1.0	2.350	472	1,110
2	D	2.35	1.0	1.0	2.350	1,490	3,500
3	A	1.58	.748	1.0	1,182	296	350
3	B	1.58	.931	1.0	1,471	387	570
3	C	1.58	.996	1.0	1.574	762	1,200
4	A	1.05	.887	1.0	.931	290	270
4	B	1.05	1.0	1.0	1.050	371	390
4	C	1.05	1.0	1.0	1.050	143	150
5	A	.68	.922	1.0	.627	638	400
5	B	.68	1.0	1.0	.680	883	600
5	C	.68	1.0	1.0	.680	3,240	2,200
6	A	.68	.807	1.0	.549	127	70
6	B	.68	.977	1.0	.664	181	120
6	C	.68	1.0	1.0	.680	279	190
7	A	.53	.803	1.0	.426	1,330	800
7	B	.53	.933	1.0	.494	1,010	500
7	C	.53	1.0	1.0	.530	566	300
7	D	.53	1.0	1.0	.530	378	200
8	A	.41	.905	1.0	.371	1,080	400
8	B	.41	1.0	1.0	.410	2,540	1,040
8	C	.41	1.0	1.0	.410	1,464	600
8	D	.41	1.0	1.0	.410	1,025	420
9	A	.41	.953	1.0	.391	102	40
9	B	.41	1.0	1.0	.410	98	40
9	C	.41	1.0	1.0	.410	171	70
9	D	.41	1.0	1.0	.410	171	70
10	A	.39	.958	1.0	.374	2,680	1,000
10	B	.39	1.0	1.0	.390	1,030	400
10	C	.39	1.0	1.0	.390	1,540	600
10	D	.39	1.0	1.0	.390	1,800	700
10	E	.39	1.0	1.0	.390	770	300
					Main River Benefits		21,420
					Tributary Benefits		21,600
					Total Benefits		43,020

TABLE 33

AVERAGE ANNUAL BENEFITS BY INDIVIDUAL RESERVOIRS

RESERVOIR	DRAINAGE AREA SQ.MI.	CAPACITY		TOTAL ANNUAL COST	FIRST IN SYSTEM							NO PREFERENCE IN SYSTEM							
		INCHES	ACRE FEET		DIRECT BENEFIT			INDIRECT BENEFIT	RESTORATION OF PROPERTY VALUES	TOTAL BENEFIT	RATIO OF BENEFIT TO COST	DIRECT BENEFIT			INDIRECT BENEFIT	RESTORATION OF PROPERTY VALUES	TOTAL BENEFIT	RATIO OF BENEFIT TO COST	
					MAIN RIVER	TRI-BUTARY	TOTAL					MAIN RIVER	TRI-BUTARY	TOTAL				(1)	(2)
Groton Pond	17.3	7.0	6,300	10,200	$ 10,310	$5,500	$ 15,810	$ 14,940	$3,870	$ 34,620	3.394	$ 6,610	$ 5,500	$ 12,110	$ 11,440	$ 3,870	$ 27,420	2.688	3.541
Lower Naukeag	19.7	5.1	5,400	28,100	7,950	28,430	36,380	34,380	21,170	91,930	3.272	4,510	23,080	27,590	26,070	21,170	74,830	2.663	3.508
Victory	66.0	7.0	24,600	37,800	33,320	17,130	50,450	47,680	12,260	110,390	2.920	22,070	13,130	35,200	33,260	12,260	80,720	2.135	2.812
Tully	50.0	8.0	16,000	36,000	16,810	23,380	40,190	37,980	19,860	98,030	2.723	9,450	18,960	28,410	26,850	19,860	75,120	2.087	2.749
N. Springfield	156.0	3.3	27,400	73,000	53,580	13,360	66,940	63,260	23,350	153,550	2.103	31,520	13,360	44,880	42,410	23,350	110,640	1.516	1.997
Birch Hill	156.3	6.0	50,000	138,700	45,940	67,670	113,610	107,360	69,630	290,600	2.095	26,280	54,980	81,260	76,790	69,630	227,680	1.642	2.163
Harvey Lake	25.0	5.9	7,800	19,800	14,940	520	15,460	14,610	5,170	35,240	1.780	9,270	520	9,790	9,250	5,170	24,210	1.223	1.611
Lyndon Center	52.0	6.0	16,600	68,000	26,110	25,510	51,620	48,780	13,290	113,690	1.672	17,600	19,560	37,160	35,120	13,290	85,570	1.258	1.657
South Branch	45.0	6.0	14,400	40,300	28,950	130	29,080	27,480	10,240	66,800	1.657	18,370	130	18,500	17,480	10,240	46,220	1.147	1.511
Surry Mountain	100.0	6.0	32,000	94,900	26,080	22,920	49,000	46,310	43,770	139,080	1.466	16,720	22,920	39,640	37,460	43,770	120,870	1.274	1.678
Gaysville	226.0	6.5	78,200	208,400	101,740	26,550	128,290	121,230	46,310	295,830	1.420	61,410	23,550	84,960	80,300	46,310	211,570	1.015	1.337
North Hartland	221.0	4.1	48,300	156,200	89,890		89,890	84,950	37,460	212,300	1.359	53,260		53,260	50,330	37,460	141,050	.903	1.190
Union Village	126.0	4.5	30,200	109,900	60,540		60,540	57,210	23,600	141,350	1.286	36,630		36,630	34,620	23,600	94,850	.863	1.137
East Haven	47.5	6.1	15,500	81,700	24,190	24,630	48,820	46,130	10,040	104,990	1.285	16,000	18,870	34,870	32,950	10,040	77,860	.953	1.255
Newfane	326.0	6.0	104,000	250,900	115,630	9,040	124,700	117,810	61,970	304,450	1.213	68,700	9,040	77,740	73,460	61,970	213,170	.850	1.120
Ayers Brook	30.0	6.0	9,600	43,400	16,000	4,730	20,730	19,590	8,190	48,510	1.118	9,470	4,200	13,670	12,920	8,190	34,780	.801	1.055
S. Tunbridge (3)	102.0	4.5	24,400	102,900	44,070	4,450	48,520	45,850	18,450	112,820	1.096	25,930	3,950	29,880	28,240	18,450	76,570	.744	.980
Knightville	164.0	4.5	39,500	113,300	25,740	10,140	35,880	33,910	40,680	110,470	.975	14,740	10,140	24,880	23,510	40,680	89,070	.786	1.035
Claremont (3)	245.0	4.5	58,600	227,200	85,270	6,580	91,850	86,800	41,740	220,390	.970	50,470	6,580	57,050	53,910	41,740	152,700	.672	.885
Bethlehem Jct. (3)	90.0	6.0	28,800	147,000	47,210	15,000	62,210	58,790	15,200	136,200	.927	28,840	15,000	43,840	41,430	15,200	100,470	.683	.900
Totals				1,987,700								527,850	263,470	791,320	747,800	526,250	2,065,370	1.039	1.369
								ALTERNATE RESERVOIRS											
Hydeville (c)	65.3	4.4	15,200	44,100	15,010	33,030	48,040	45,400	29,000	122,440	2.776	9,920	27,200	37,120	35,080	29,000	101,200	2.294	3.022
Priest Pond (d)	19.0	6.0	6,000	27,300	4,920	9,660	14,580	13,780	8,350	36,710	1.345	3,270	7,940	11,210	10,590	8,350	30,150	1.104	1.454
Stocker Pond (e)	35.0	6.0	11,200	30,500	12,610	4,130	16,740	15,820	5,950	38,510	1.263	8,510	4,130	12,640	11,940	5,950	30,530	1.001	1.319
Mascoma Lake	73.0	4.4	17,000	57,400	19,060	8,350	27,410	25,900	12,400	65,710	1.145	12,960	6,410	19,370	18,300	12,400	50,070	.872	1.149
Gale River	86.0	2.9	13,400	61,000	21,290	6,130	27,420	25,910	8,450	61,780	1.013	16,100	5,080	21,180	20,020	8,450	49,650	.814	1.072
Centerville (f)	692.0	4.2	154,600	540,300	244,030		244,030	230,610	11,600	486,240	.900	167,570		167,570	158,350	11,600	337,520	.625	.823
Perkinsville (b)	142.0	6.0	45,400	193,100	60,200	16,590	76,790	72,570	23,350	172,710	.894	39,850	16,590	56,440	53,340	23,350	133,130	.689	.908
West Canaan	80.0	6.0	25,700	104,800	27,300	11,080	38,380	36,270	13,600	88,250	.842	18,790	8,510	27,300	25,800	13,600	66,700	.636	.838
Ludlow (a)	56.0	4.5	13,400	86,000	19,510	14,270	33,780	31,920	2,800	68,500	.797	13,710	12,990	26,700	25,230	2,800	54,730	.636	.838
Bath	397.0	6.0	127,000	518,500	148,210		148,210	140,060	61,800	350,070	.675	108,350		108,350	102,390	61,800	272,540	.526	.693

(1) Based on estimated annual costs per Paragraph 81 of the Report.

(2) Annual costs adjusted per Paragraph 139 of the Report.

(3) When consideration is given to unevaluated benefits and the fact that these reservoirs are needed to develop the full value of the other reservoirs, it is considered that their benefits in fact exceed their costs.

(a) After North Springfield.

(b) Alternate to North Springfield.

(c)
(d) Alternate to Birch Hill.

(e) If Claremont is not constructed.

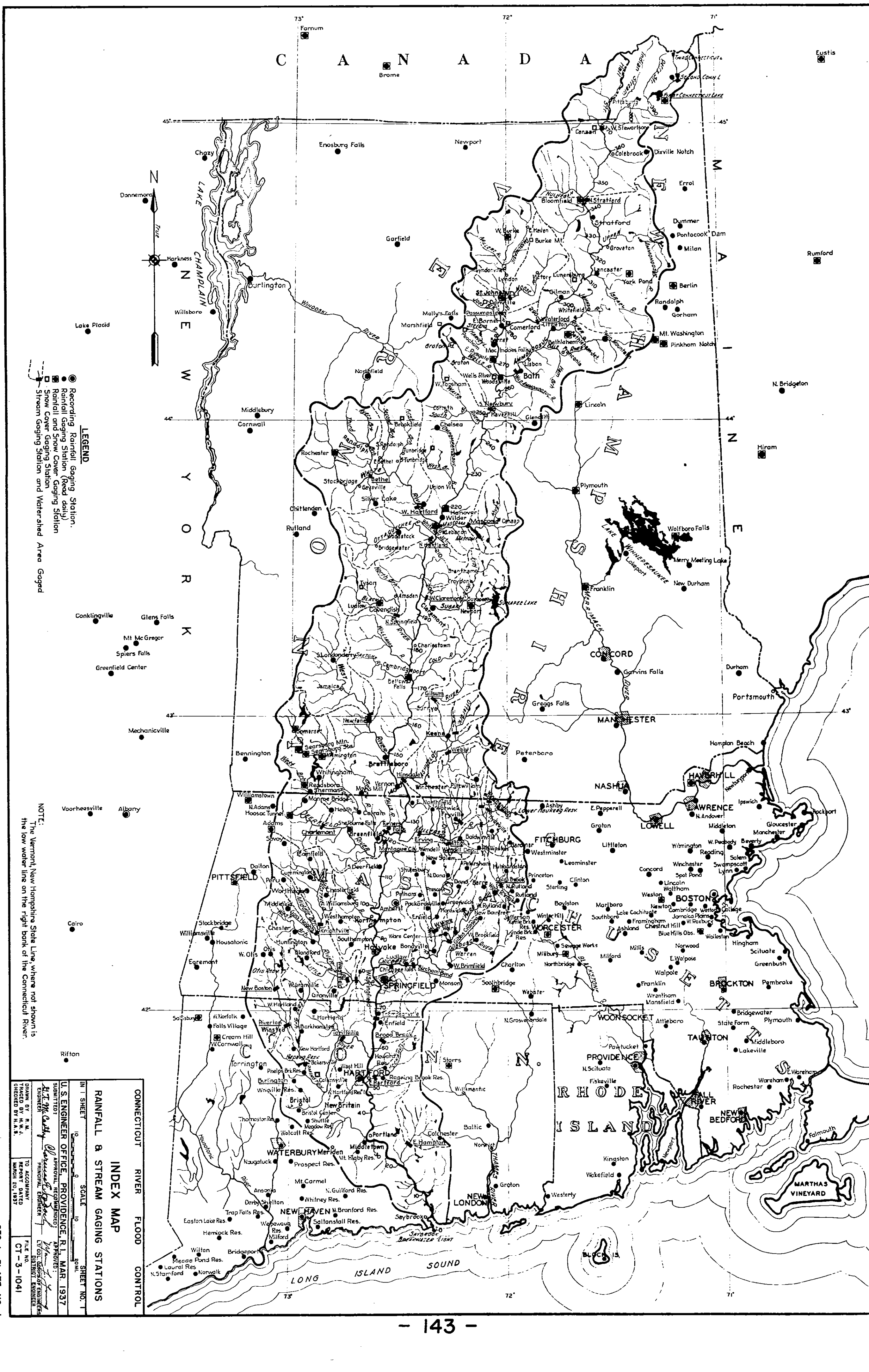
(f) Alternate to other White River Reservoirs.

CT-9-1083

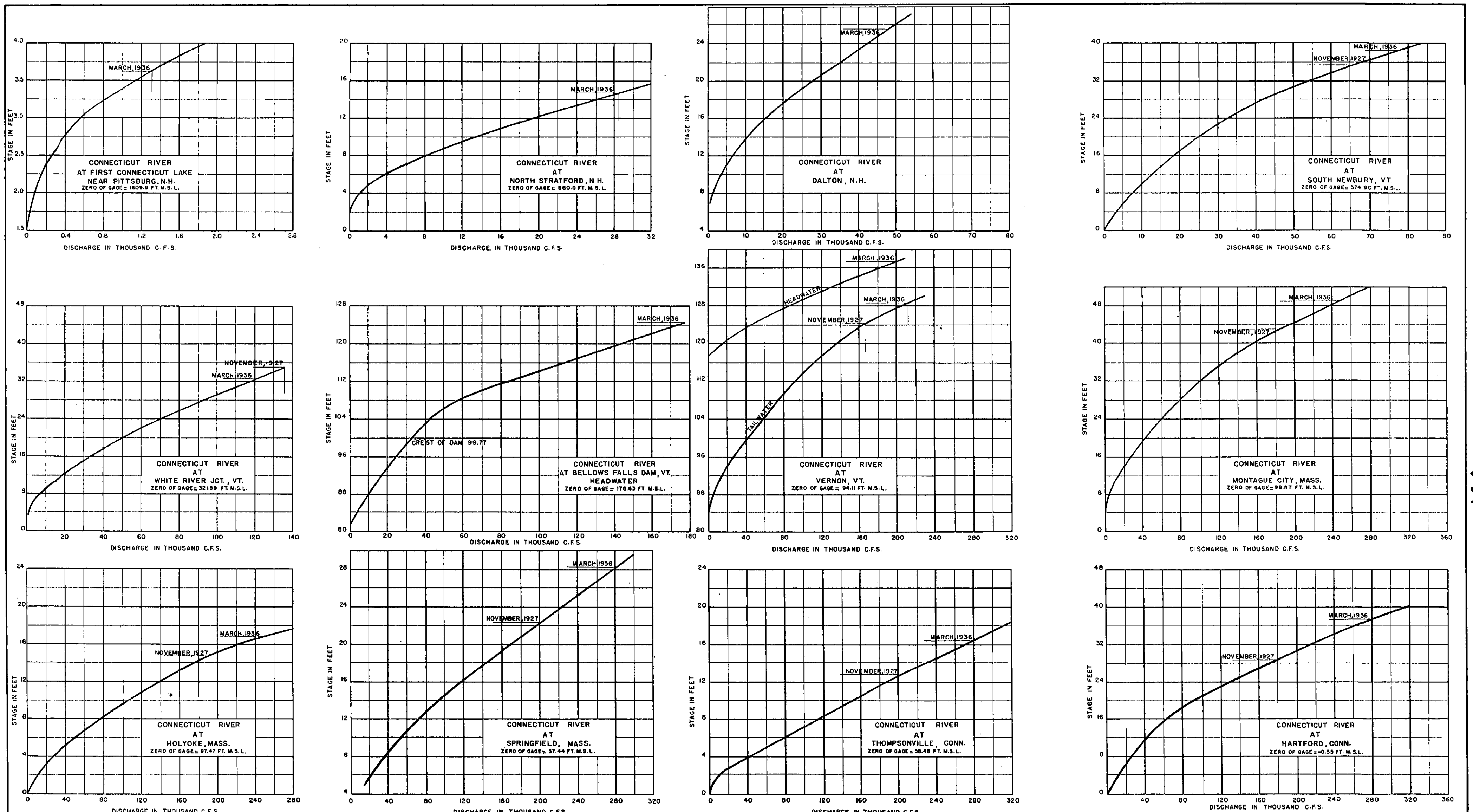
SECTION I

PLATE REFERENCE

WAR DEPARTMENT



CORPS OF ENGINEERS, U. S. ARMY



CONNECTICUT RIVER FLOOD CONTROL

CONNECTICUT RIVER

RATING CURVES

IN 2 SHEETS | SCALES AS SHOWN | SHEET NO. 1

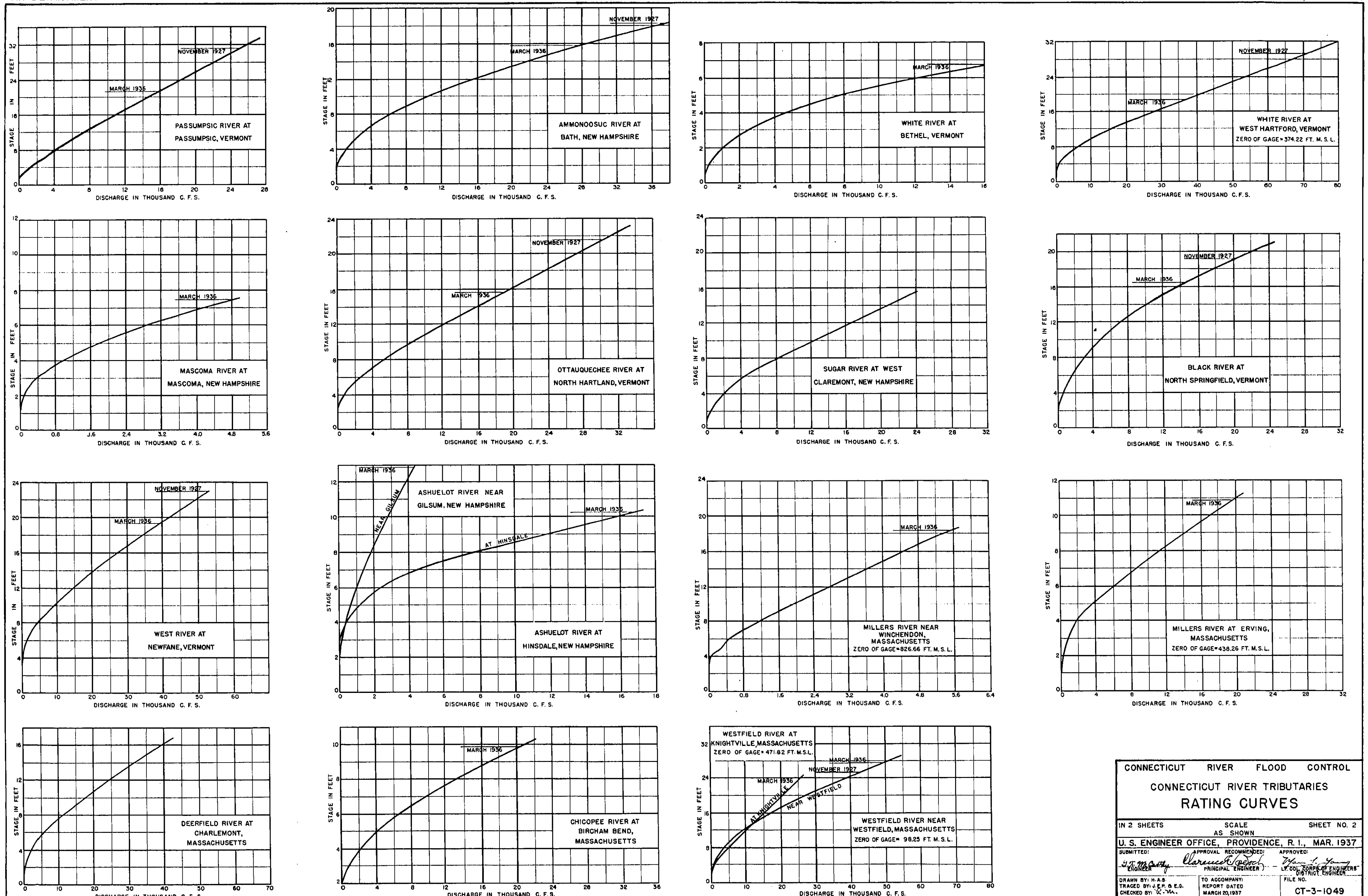
U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: G. T. McCarthy, ENGINEER
APPROVAL RECOMMENDED: Clarence E. Kerbel, PRINCIPAL ENGINEER
APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: H.A.S.
TRACED BY: J.E.P.-J.F.J.
CHECKED BY: R.W.

TO ACCOMPANY REPORT DATED MARCH 20, 1937

FILE NO. CT-3-1048



CONNECTICUT RIVER FLOOD CONTROL

CONNECTICUT RIVER TRIBUTARIES

RATING CURVES

IN 2 SHEETS — SCALE AS SHOWN — SHEET NO. 2

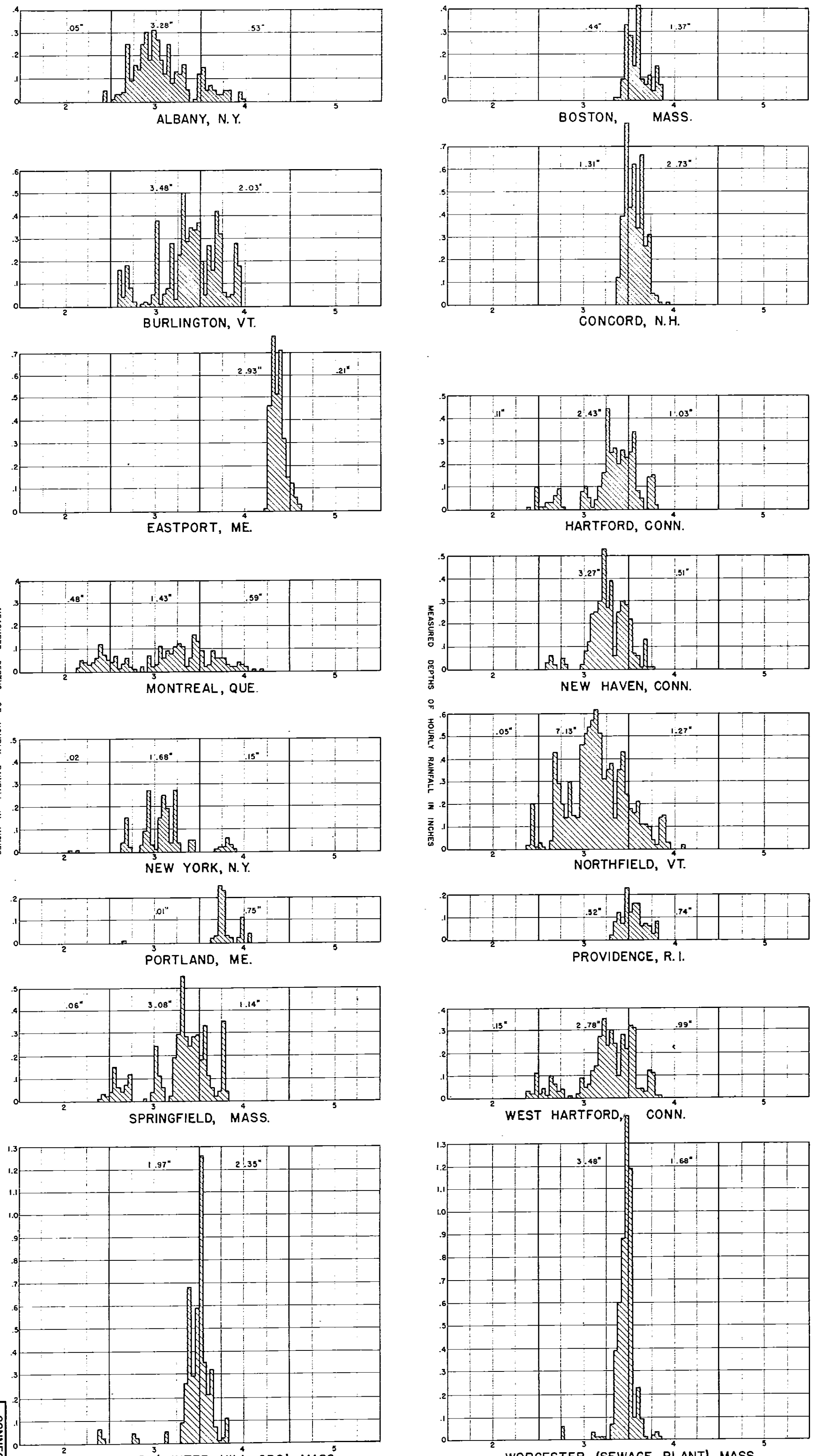
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MAR. 1937

SUBMITTED: G. T. McCarthy, ENGINEER — APPROVAL RECOMMENDED: Clarence Spelch, PRINCIPAL ENGINEER — APPROVED: Wm. L. Young, LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: H.A.B.
TRACED BY: J.F.P. & E.G.
CHECKED BY: R.-M.

TO ACCOMPANY: REPORT DATED MARCH 20, 1937

FILE NO. CT-3-1049



CONNECTICUT RIVER FLOOD CONTROL
NORTHEASTERN UNITED STATES
HOURLY RAINFALL RECORDS
FOR
NOVEMBER 2-5, 1927

IN 1 SHEET | SCALE AS SHOWN | SHEET NO. 1

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR. 1937

SUBMITTED: ENGINEER | APPROVAL RECOMMENDED: PRINCIPAL ENGINEER | APPROVED:

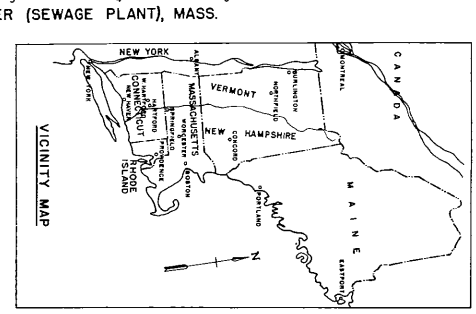
DRAWN BY: W.G.N. TRACED BY: H.M.J. CHECKED BY:

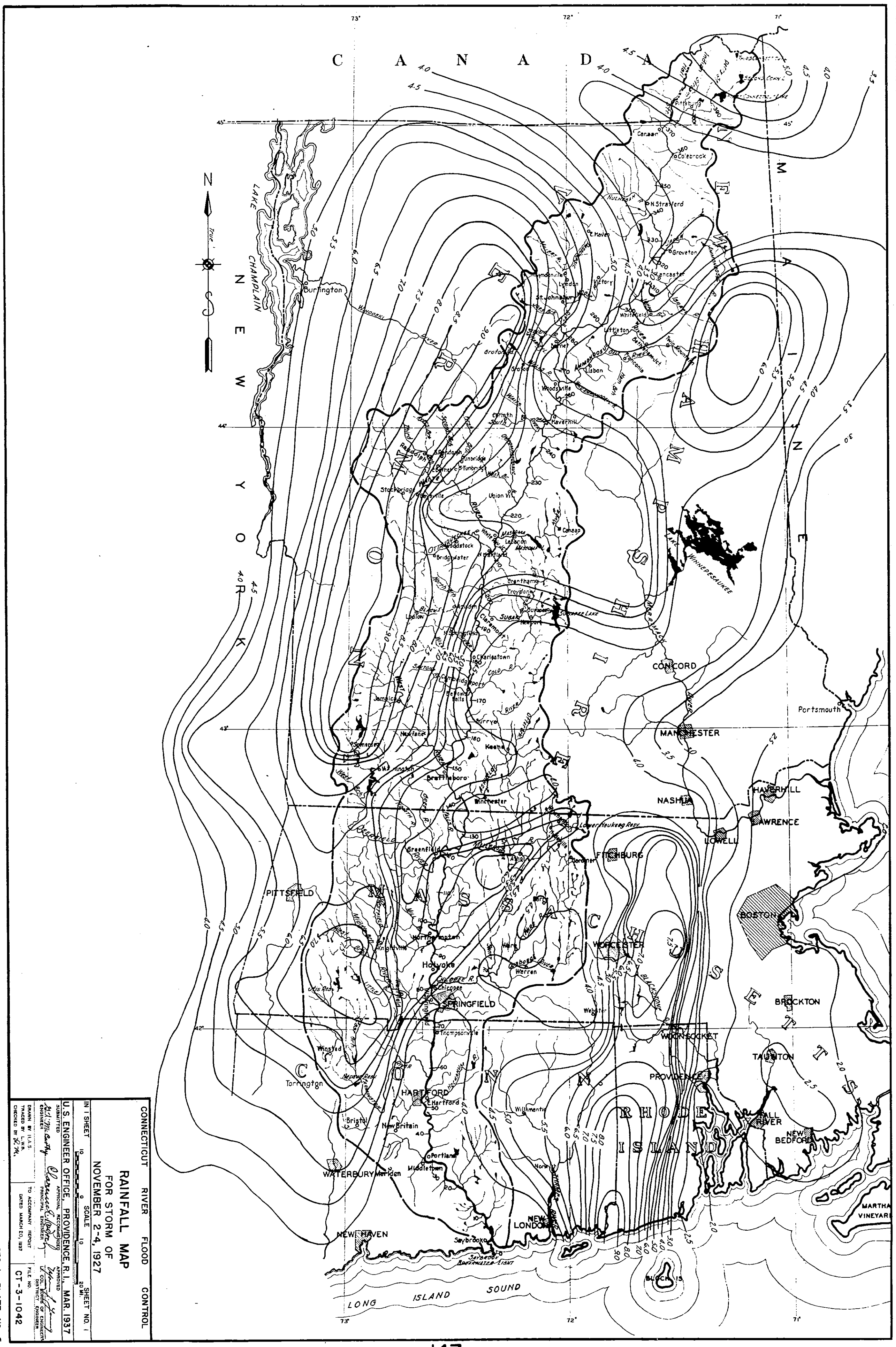
TO ACCOMPANY REPORT DATED: MARCH 20, 1937

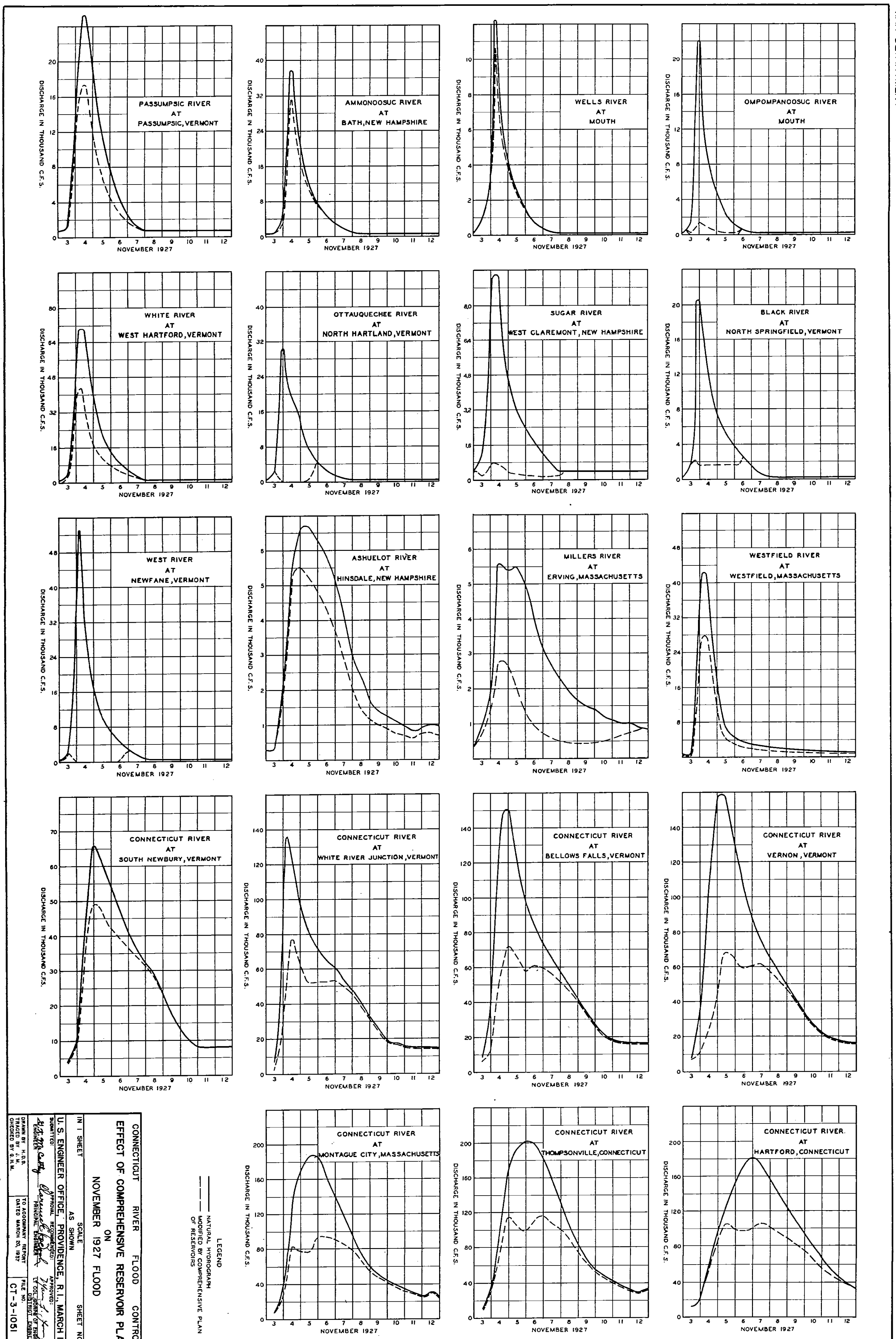
FILE NO. CT-3-1050

NOTES:
DATA FROM UNITED STATES WEATHER BUREAU.
TOTAL DEPTH OF RAINFALL IN INCHES FOR EACH DAY ARE SHOWN ABOVE THE GRAPHS.
DAYS ARE FROM MIDNIGHT TO MIDNIGHT.

VICINITY MAP



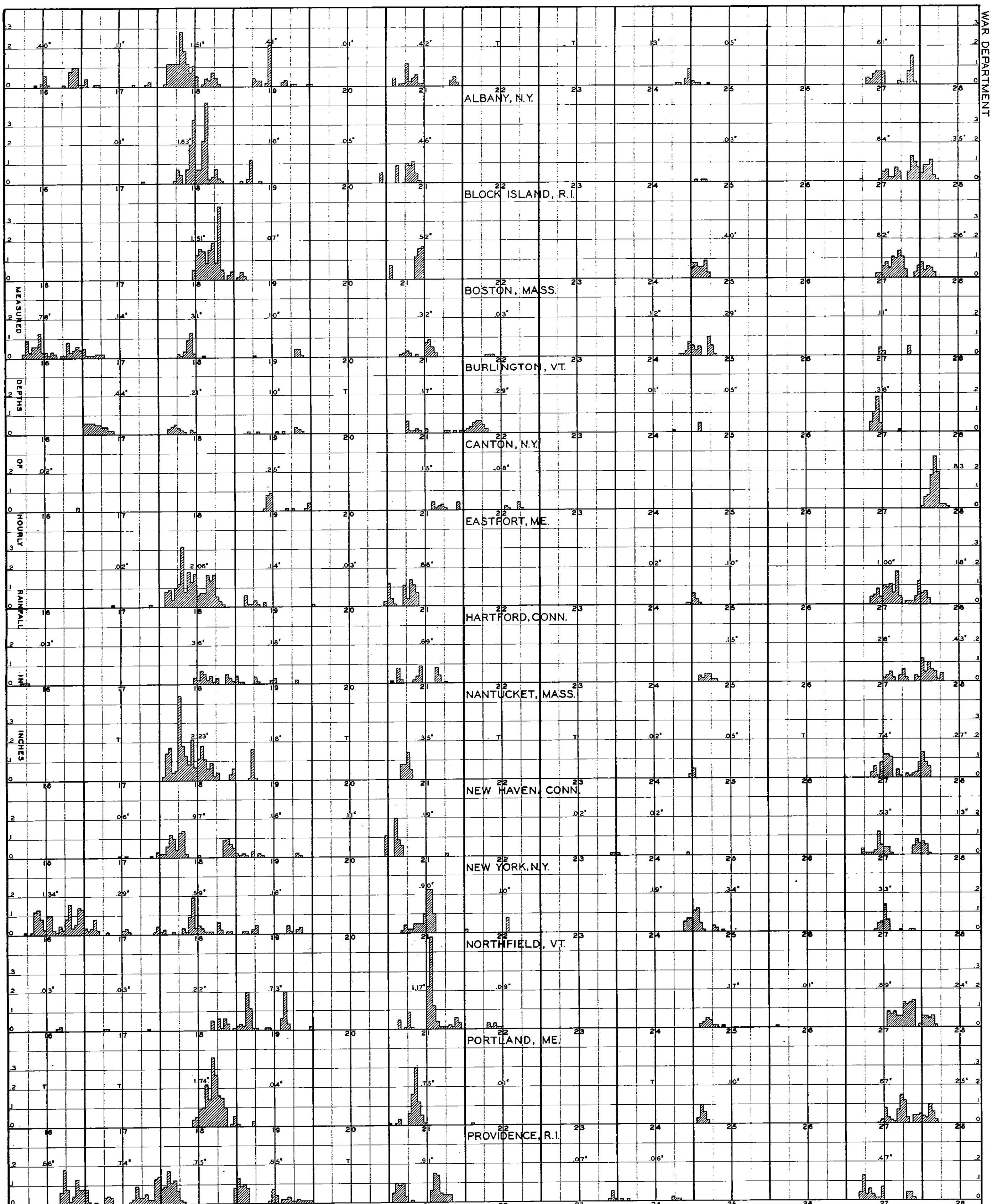

WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
CONNECTICUT RIVER FLOOD CONTROL
RAINFALL MAP
FOR STORM OF
NOVEMBER 2-4, 1927
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR. 1937
IN 1 SHEET
SHEET NO. 1
CT-3-1042
CANADA
NEW YORK
VERMONT
NEW HAMPSHIRE
MAINE
MASSACHUSETTS
CONNECTICUT
RHODE ISLAND
LAKE CHAMPLAIN
LAKE WINNEPESAUKEE
LONG ISLAND SOUND
Burlington
CONCORD
MANCHESTER
NASHUA
Portsmouth
HAVERHILL
LAWRENCE
LOWELL
FITCHBURG
BOSTON
WORCESTER
BROCKTON
TAUNTON
PROVIDENCE
FALL RIVER
NEW BEDFORD
PITTSFIELD
SPRINGFIELD
Holyoke
Northampton
HARTFORD
New Britain
Bristol
Torrington
Winsted
WATERBURY
Meriden
Middletown
NEW HAVEN
NEW LONDON
Saybrook
BLOCK IS.
Greenfield
Brattleboro
Keene
Claremont
St. Johnsbury
Lyndon
Littleton
Lancaster
Colebrook
Woodsville
White River Jct.
Windsor
Ludlow
Jamaica
Wilmington
Winchester
Ware
Warren
Webster
Woonsocket
Willimantic
Norwich



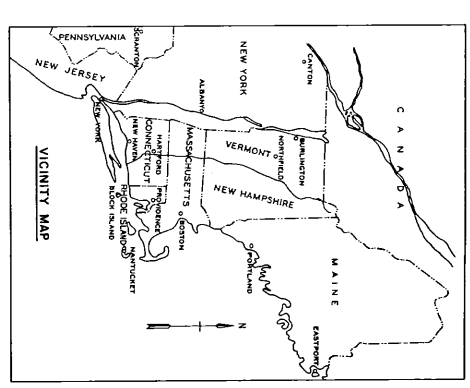
WAR DEPARTMENT

CORPS OF ENGINEERS U.S. ARMY

WAR DEPARTMENT



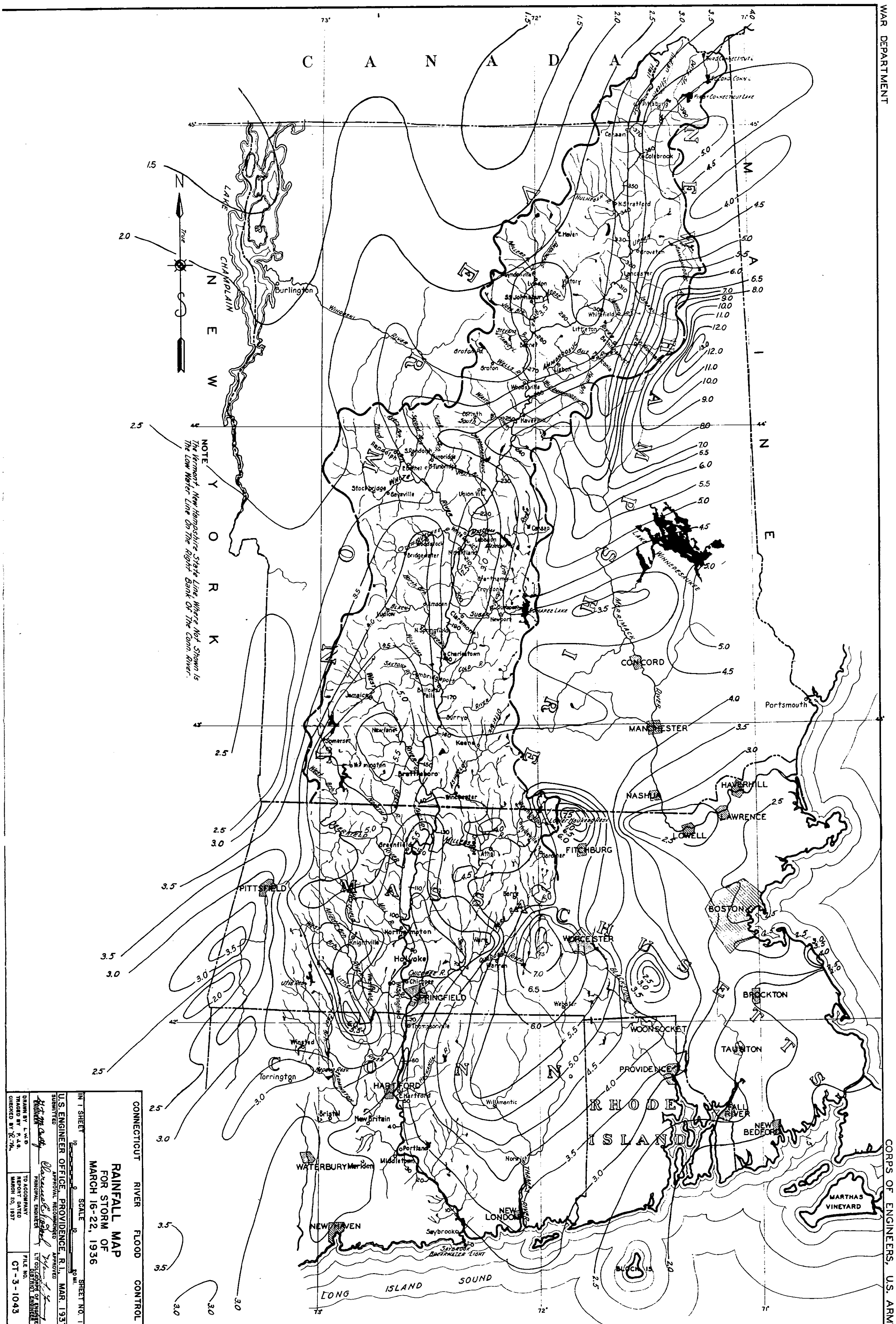
NOTES:
DATA FROM UNITED STATES WEATHER BUREAU.
TOTAL DEPTH OF RAINFALL IN INCHES FOR EACH DAY ARE SHOWN ABOVE THE GRAPHS.
DAYS ARE FROM MIDNIGHT TO MIDNIGHT.

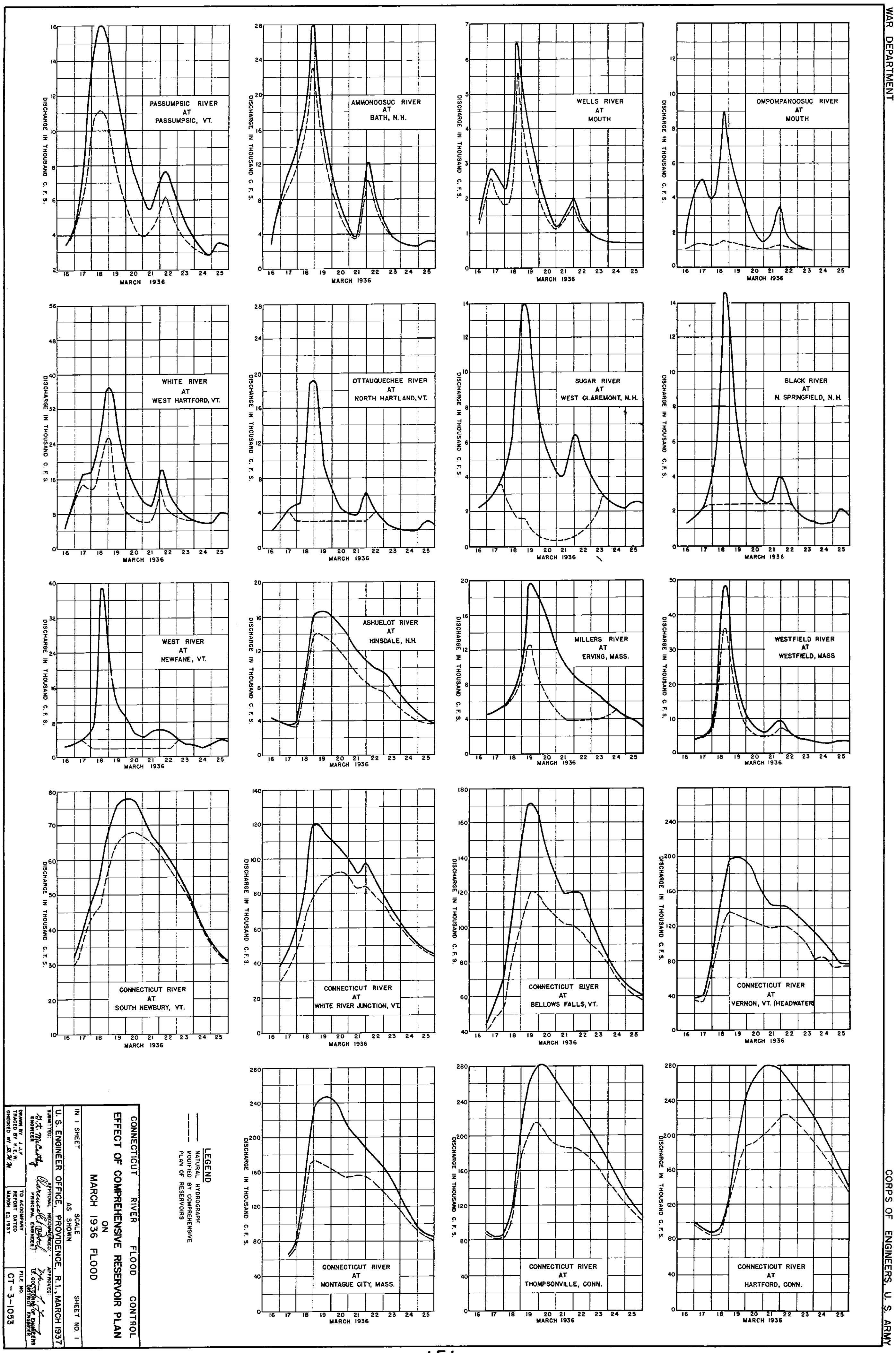


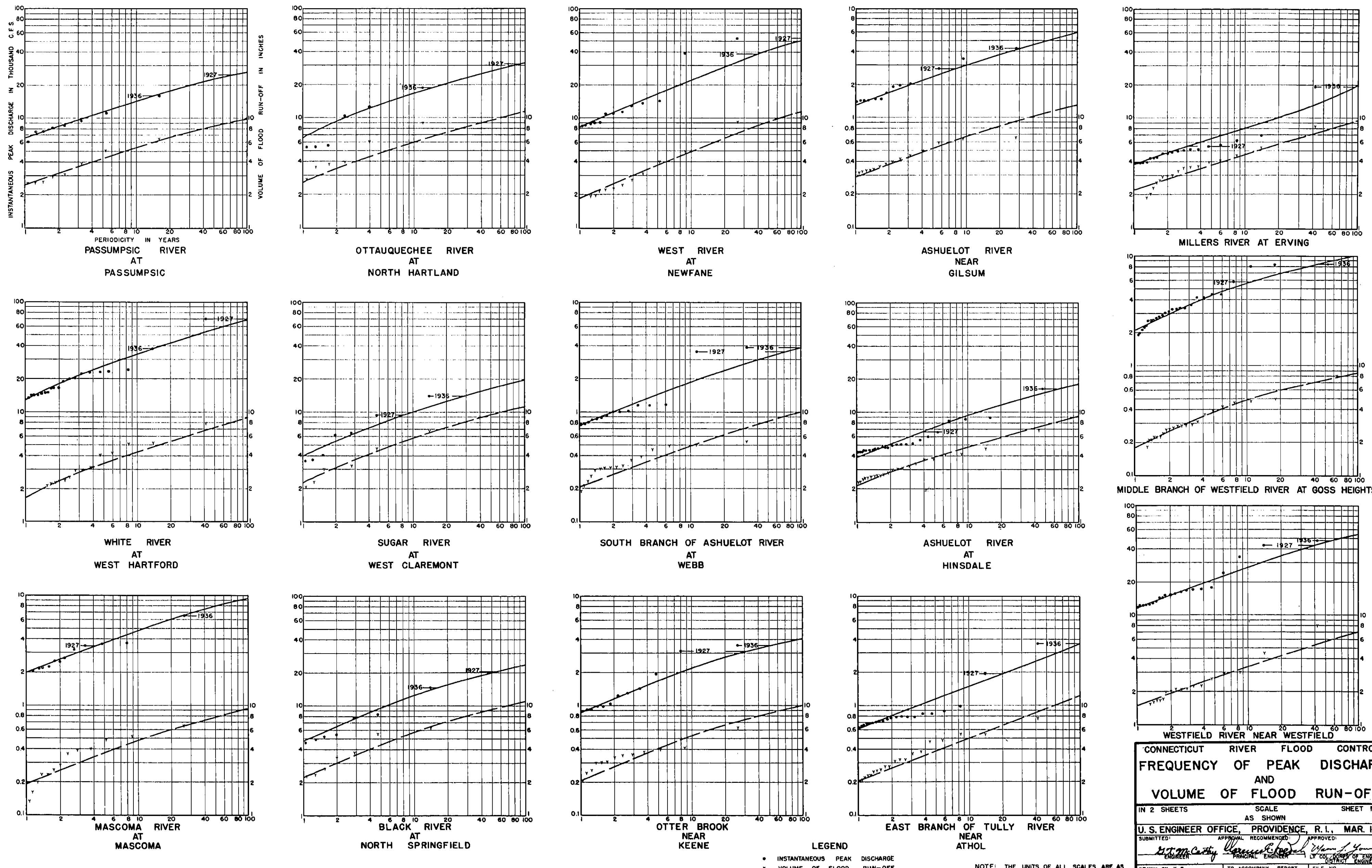
CONNECTICUT RIVER FLOOD CONTROL
NORTHEASTERN UNITED STATES
HOURLY RAINFALL RECORDS
FOR
MARCH 16-28, 1936

IN 1 SHEET	SHEET NO. 1
SCALE AS SHOWN	
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937	
SUBMITTED: ENGINEER	APPROVAL RECOMMENDED: PRINCIPAL ENGINEER / APPROVED: LT. COL. CORPS OF ENGINEERS DISTRICT ENGINEER
DRAWN BY W.O.N. TRACED BY W.J.M. CHECKED BY R.M.	TO ACCOMPANY REPORT DATED MARCH 20, 1937 / FILE NO. CT-3-1052

CORPS OF ENGINEERS, U.S. ARMY


WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
CONNECTICUT RIVER FLOOD CONTROL
RAINFALL MAP
FOR STORM OF
MARCH 16-22, 1936
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR. 1937
CT-3-1043
NOTE
The Vermont, New Hampshire State Line, Where Not Shown Is The Low Water Line On The Right Bank Of The Conn. River.
CANADA
NEW YORK
VERMONT
NEW HAMPSHIRE
MAINE
MASSACHUSETTS
CONNECTICUT
RHODE ISLAND
LAKE CHAMPLAIN
LAKE WINNEPESAUKEE
LONG ISLAND SOUND
MARTHAS VINEYARD
BLOCK IS.
Burlington
CONCORD
MANCHESTER
NASHUA
LOWELL
LAWRENCE
HAVERHILL
Portsmouth
FITCHBURG
BOSTON
WORCESTER
BROCKTON
TAUNTON
WOONSOCKET
PROVIDENCE
FALL RIVER
NEW BEDFORD
PITTSFIELD
SPRINGFIELD
Holyoke
Northampton
HARTFORD
New Britain
WATERBURY
Meriden
Middletown
NEW HAVEN
NEW LONDON
Torrington
Winsted
Bristol
Saybrook
Brattleboro
Keene
Claremont
Newport
Colebrook
Lancaster
Littleton
St. Johnsbury
Woodstock


WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY
PASSUMPSIC RIVER AT PASSUMPSIC, VT.
AMMONOOSUC RIVER AT BATH, N.H.
WELLS RIVER AT MOUTH
OMPOMPANOOSUC RIVER AT MOUTH
WHITE RIVER AT WEST HARTFORD, VT.
OTTAUQUECHEE RIVER AT NORTH HARTLAND, VT.
SUGAR RIVER AT WEST CLAREMONT, N.H.
BLACK RIVER AT N. SPRINGFIELD, N.H.
WEST RIVER AT NEWFANE, VT.
ASHUELOT RIVER AT HINSDALE, N.H.
MILLERS RIVER AT ERVING, MASS.
WESTFIELD RIVER AT WESTFIELD, MASS
CONNECTICUT RIVER AT SOUTH NEWBURY, VT.
CONNECTICUT RIVER AT WHITE RIVER JUNCTION, VT.
CONNECTICUT RIVER AT BELLOWS FALLS, VT.
CONNECTICUT RIVER AT VERNON, VT. (HEADWATER)
CONNECTICUT RIVER AT MONTAGUE CITY, MASS.
CONNECTICUT RIVER AT THOMPSONVILLE, CONN.
CONNECTICUT RIVER AT HARTFORD, CONN.
DISCHARGE IN THOUSAND C.F.S.
MARCH 1936
LEGEND
NATURAL HYDROGRAPH
MODIFIED BY COMPREHENSIVE PLAN OF RESERVOIRS
CONNECTICUT RIVER FLOOD CONTROL
EFFECT OF COMPREHENSIVE RESERVOIR PLAN
ON
MARCH 1936 FLOOD
IN 1 SHEET
SCALE AS SHOWN
SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1937
FILE NO. CT-3-1053



LEGEND
- • INSTANTANEOUS PEAK DISCHARGE
- Y VOLUME OF FLOOD RUN-OFF

NOTE: THE UNITS OF ALL SCALES ARE AS SHOWN IN THE EXAMPLE IN THE UPPER LEFT HAND CORNER.

CONNECTICUT RIVER FLOOD CONTROL

FREQUENCY OF PEAK DISCHARGE AND VOLUME OF FLOOD RUN-OFF

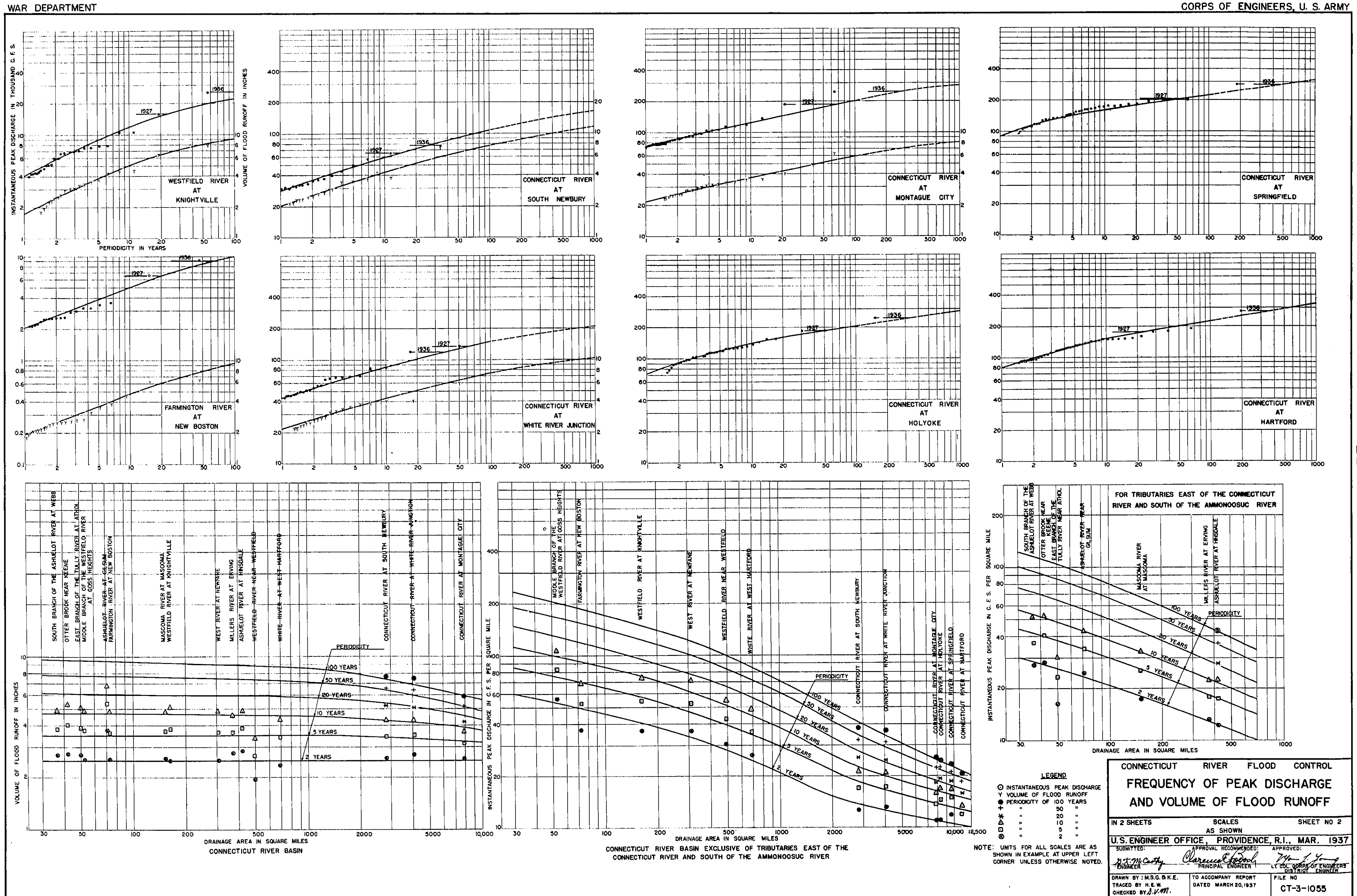
IN 2 SHEETS | SCALE AS SHOWN | SHEET NO. 1

U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MAR. 1937

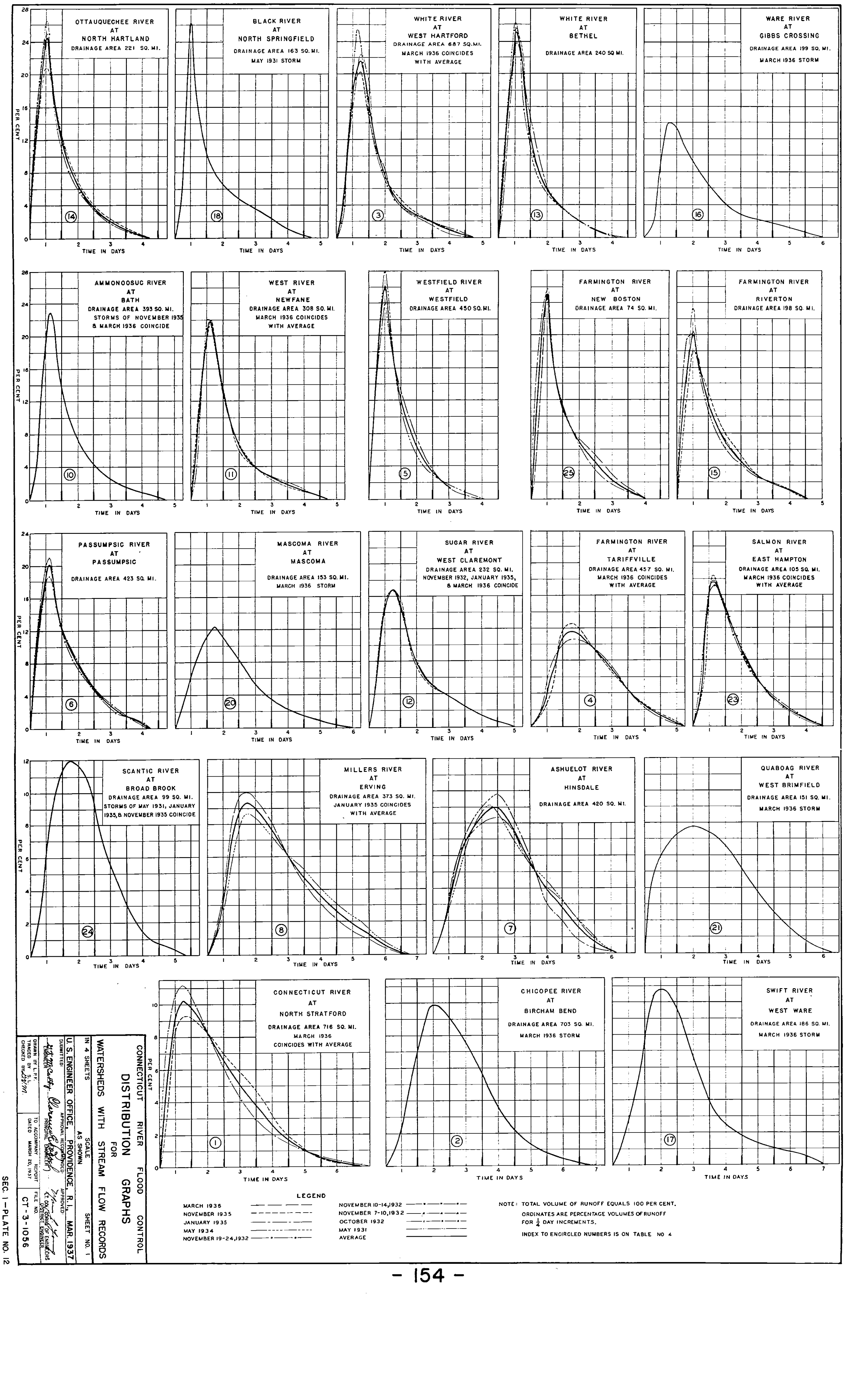
SUBMITTED: G. T. McCarthy, ENGINEER | APPROVAL RECOMMENDED: PRINCIPAL ENGINEER | APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY K. E. TRACED BY E.S.W. CHECKED BY D.T.M. | TO ACCOMPANY REPORT DATED MARCH 20, 1937 | FILE NO. CT-3-1054

SEC. I—PLATE NO. 10



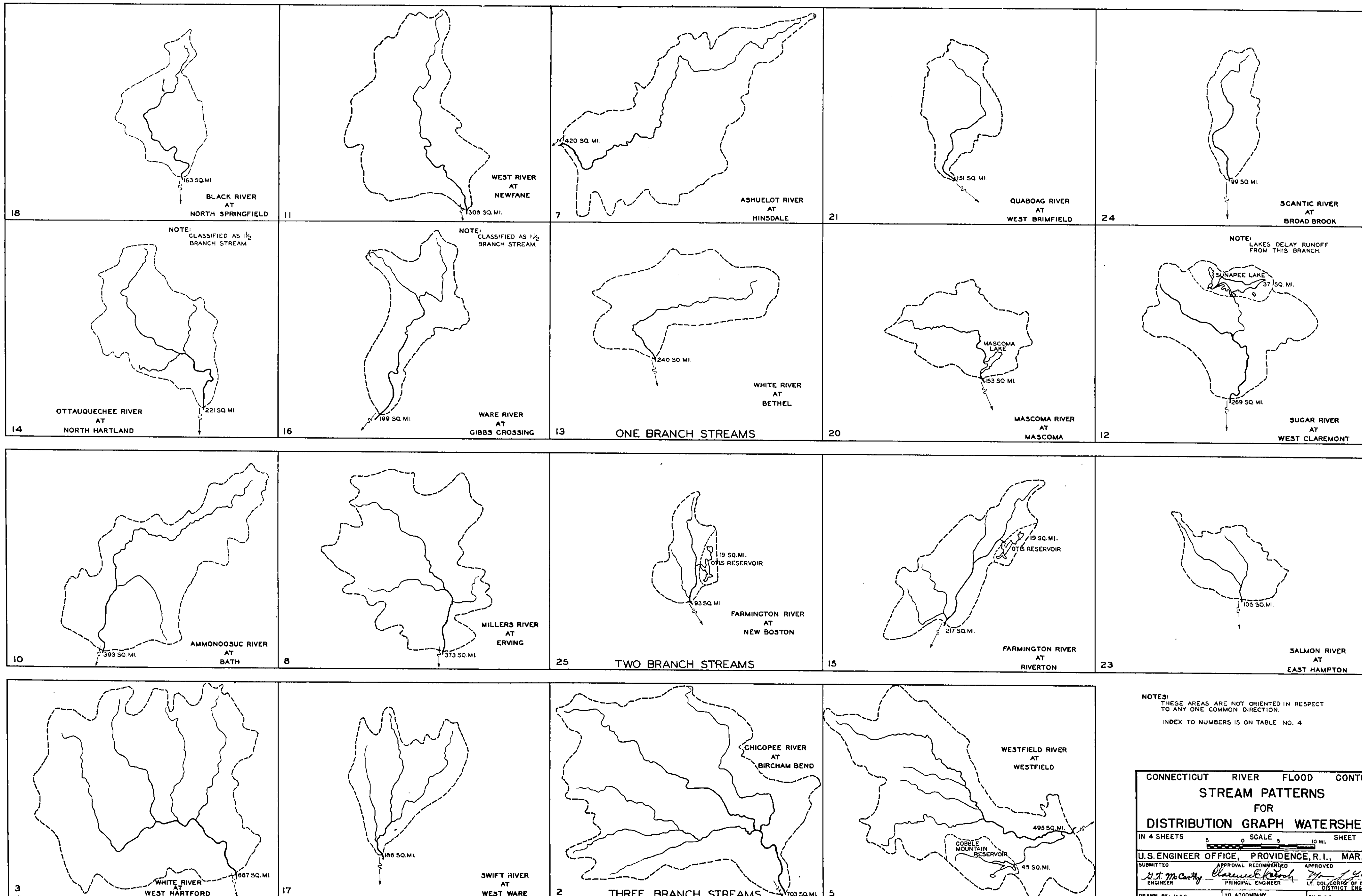
SEC. 1—PLATE NO. 11



OTTAUQUECHEE RIVER AT NORTH HARTLAND DRAINAGE AREA 221 SQ. MI.
BLACK RIVER AT NORTH SPRINGFIELD DRAINAGE AREA 163 SQ. MI. MAY 1931 STORM
WHITE RIVER AT WEST HARTFORD DRAINAGE AREA 687 SQ. MI. MARCH 1936 COINCIDES WITH AVERAGE
WHITE RIVER AT BETHEL DRAINAGE AREA 240 SQ. MI.
WARE RIVER AT GIBBS CROSSING DRAINAGE AREA 199 SQ. MI. MARCH 1936 STORM
AMMONOOSUC RIVER AT BATH DRAINAGE AREA 393 SQ. MI. STORMS OF NOVEMBER 1935 & MARCH 1936 COINCIDE
WEST RIVER AT NEWFANE DRAINAGE AREA 308 SQ. MI. MARCH 1936 COINCIDES WITH AVERAGE
WESTFIELD RIVER AT WESTFIELD DRAINAGE AREA 450 SQ. MI.
FARMINGTON RIVER AT NEW BOSTON DRAINAGE AREA 74 SQ. MI.
FARMINGTON RIVER AT RIVERTON DRAINAGE AREA 198 SQ. MI.
PASSUMPSIC RIVER AT PASSUMPSIC DRAINAGE AREA 423 SQ. MI.
MASCOMA RIVER AT MASCOMA DRAINAGE AREA 153 SQ. MI. MARCH 1936 STORM
SUGAR RIVER AT WEST CLAREMONT DRAINAGE AREA 232 SQ. MI. NOVEMBER 1932, JANUARY 1935, & MARCH 1936 COINCIDE
FARMINGTON RIVER AT TARIFFVILLE DRAINAGE AREA 457 SQ. MI. MARCH 1936 COINCIDES WITH AVERAGE
SALMON RIVER AT EAST HAMPTON DRAINAGE AREA 105 SQ. MI. MARCH 1936 COINCIDES WITH AVERAGE
SCANTIC RIVER AT BROAD BROOK DRAINAGE AREA 99 SQ. MI. STORMS OF MAY 1931, JANUARY 1935, & NOVEMBER 1935 COINCIDE
MILLERS RIVER AT ERVING DRAINAGE AREA 373 SQ. MI. JANUARY 1935 COINCIDES WITH AVERAGE
ASHUELOT RIVER AT HINSDALE DRAINAGE AREA 420 SQ. MI.
QUABOAG RIVER AT WEST BRIMFIELD DRAINAGE AREA 151 SQ. MI. MARCH 1936 STORM
CONNECTICUT RIVER AT NORTH STRATFORD DRAINAGE AREA 716 SQ. MI. MARCH 1936 COINCIDES WITH AVERAGE
CHICOPEE RIVER AT BIRCHAM BEND DRAINAGE AREA 703 SQ. MI. MARCH 1936 STORM
SWIFT RIVER AT WEST WARE DRAINAGE AREA 186 SQ. MI. MARCH 1936 STORM
PER CENT
TIME IN DAYS
LEGEND
MARCH 1936
NOVEMBER 1935
JANUARY 1935
MAY 1934
NOVEMBER 19-24, 1932
NOVEMBER 10-14, 1932
NOVEMBER 7-10, 1932
OCTOBER 1932
MAY 1931
AVERAGE
NOTE: TOTAL VOLUME OF RUNOFF EQUALS 100 PER CENT. ORDINATES ARE PERCENTAGE VOLUMES OF RUNOFF FOR 1/4 DAY INCREMENTS. INDEX TO ENCIRCLED NUMBERS IS ON TABLE NO. 4
CONNECTICUT RIVER FLOOD CONTROL
DISTRIBUTION GRAPHS FOR WATERSHEDS WITH STREAM FLOW RECORDS
IN 4 SHEETS SCALE AS SHOWN SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937
TO ACCOMPANY REPORT DATED MARCH 20, 1937
CT-3-1056
WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
SEC. I - PLATE NO. 12

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



NOTES:
THESE AREAS ARE NOT ORIENTED IN RESPECT TO ANY ONE COMMON DIRECTION.
INDEX TO NUMBERS IS ON TABLE NO. 4

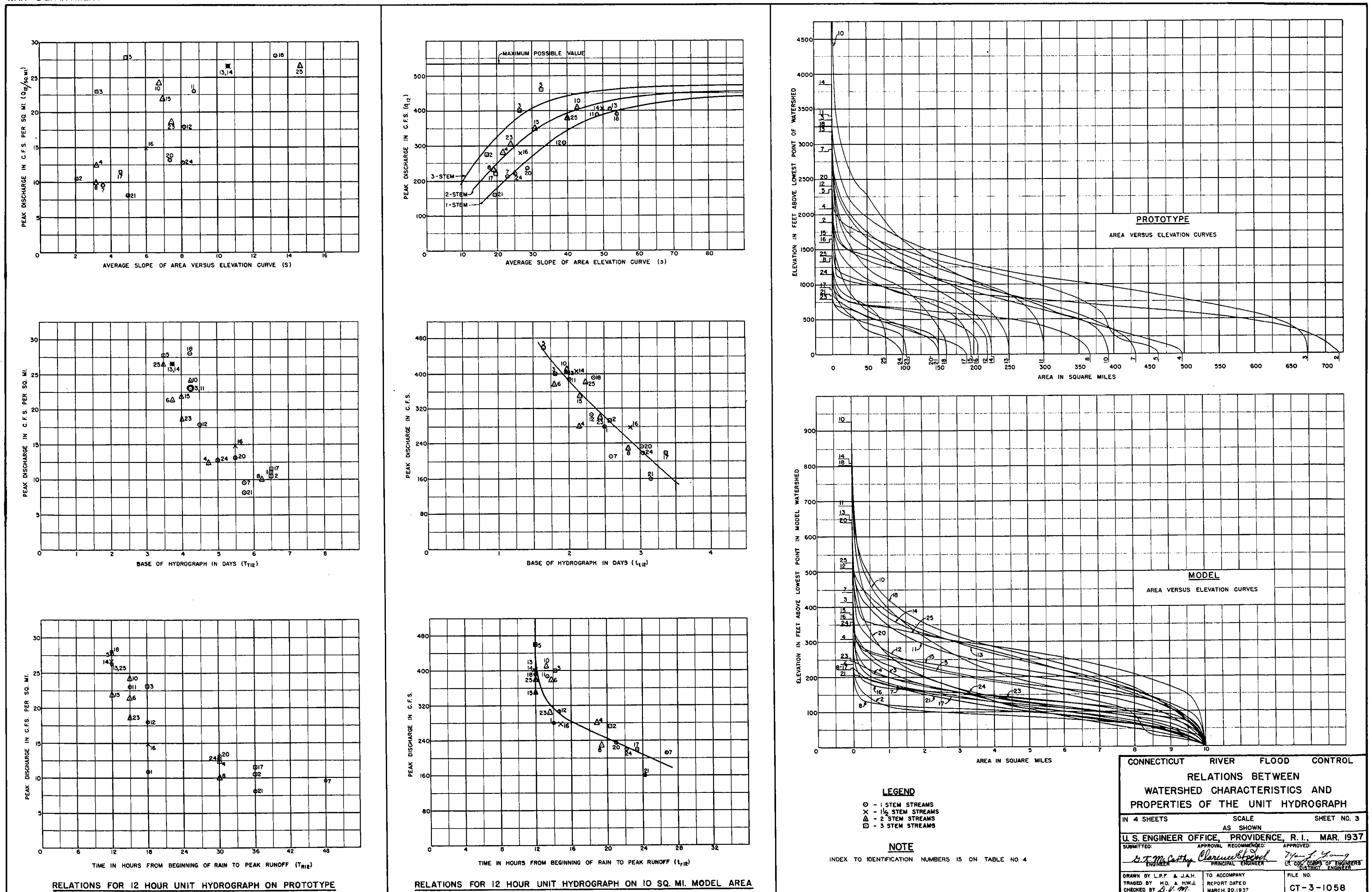
CONNECTICUT RIVER FLOOD CONTROL
STREAM PATTERNS
FOR
DISTRIBUTION GRAPH WATERSHEDS

IN 4 SHEETS — SCALE 5 0 5 10 MI. — SHEET NO 2

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: G. T. McCarthy, ENGINEER
APPROVAL RECOMMENDED: Clarence E. Ketchum, PRINCIPAL ENGINEER
APPROVED: Wm. J. Young, LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: M.S.G.
TRACED BY: H.O.
CHECKED BY: D.V.M.
TO ACCOMPANY REPORT DATED MARCH 20, 1937
FILE NO. CT-3-1045



RELATIONS FOR 12 HOUR UNIT HYDROGRAPH ON PROTOTYPE

RELATIONS FOR 12 HOUR UNIT HYDROGRAPH ON 10 SQ. MI. MODEL AREA

LEGEND

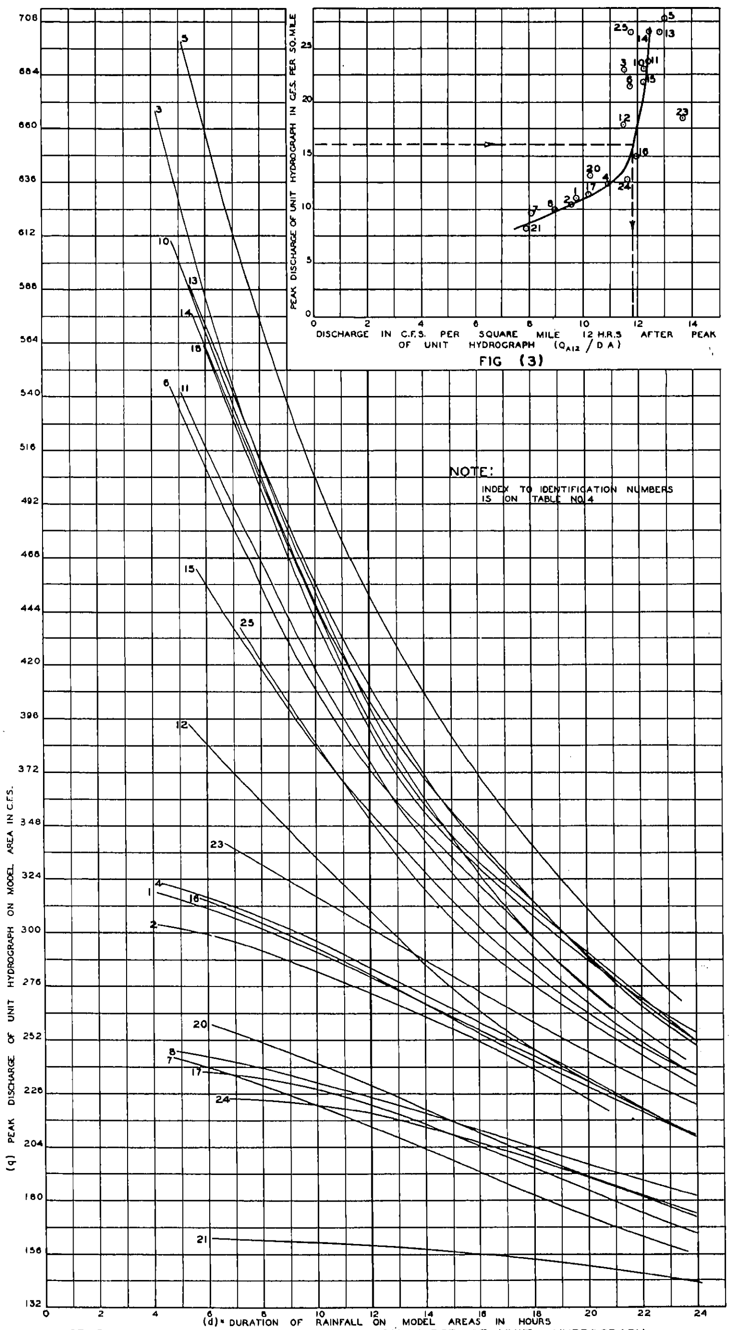
- ⊖ - 1 STEM STREAMS
- × - 1½ STEM STREAMS
- △ - 2 STEM STREAMS
- ⊡ - 3 STEM STREAMS

NOTE

INDEX TO IDENTIFICATION NUMBERS IS ON TABLE NO. 4

CONNECTICUT RIVER FLOOD CONTROL RELATIONS BETWEEN WATERSHED CHARACTERISTICS AND PROPERTIES OF THE UNIT HYDROGRAPH	
IN 4 SHEETS	SCALE AS SHOWN — SHEET NO. 3
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.	MAR. 1937
SUBMITTED: G. T. McCarthy, ENGINEER	APPROVAL RECOMMENDED: Clarence ..., PRINCIPAL ENGINEER — APPROVED: ... LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER
DRAWN BY L.P.F. & J.A.H. TRACED BY H.O. & H.W.J. CHECKED BY D.V.M.	TO ACCOMPANY REPORT DATED MARCH 20, 1937 — FILE NO. CT-3-1058

SEC. I PLATE NO. 14



GRAPH A RELATION BETWEEN PEAK DISCHARGE OF UNIT HYDROGRAPH AND DURATION OF RAINFALL FOR MODEL AREA OF 10 SQ. MILES.

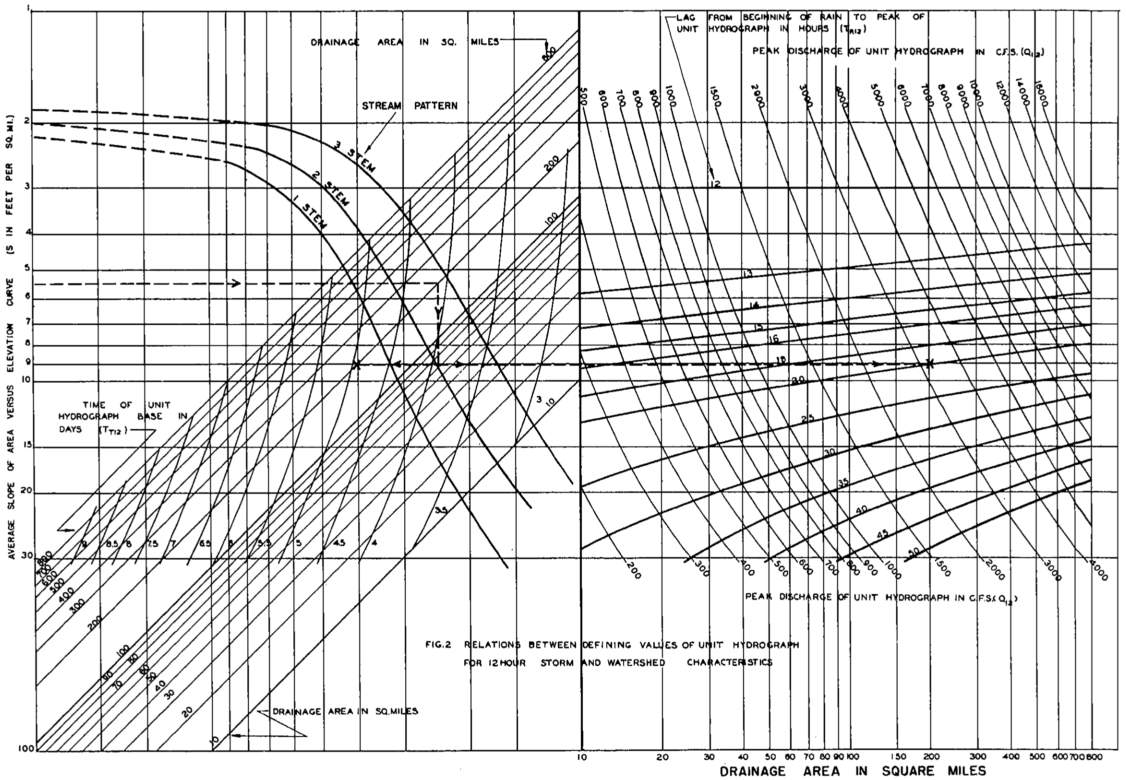


FIG. 2 RELATIONS BETWEEN DEFINING VALUES OF UNIT HYDROGRAPH FOR 12 HOUR STORM AND WATERSHED CHARACTERISTICS

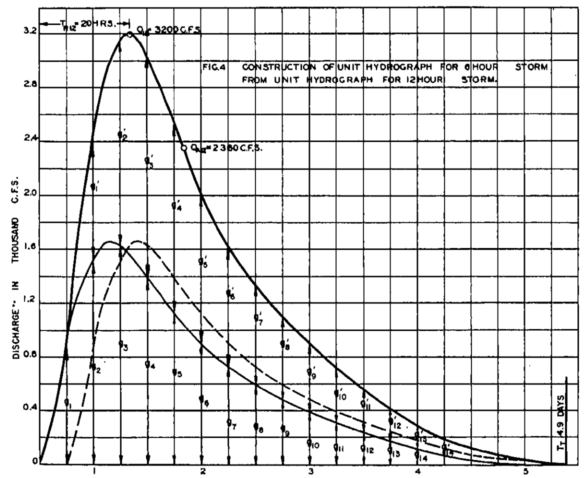


FIG. 4 CONSTRUCTION OF UNIT HYDROGRAPH FOR 6 HOUR STORM FROM UNIT HYDROGRAPH FOR 12 HOUR STORM.

EXAMPLE:

GIVEN:

S = 5.5 FEET PER SQUARE MILE.
DRAINAGE AREA = 200 SQUARE MILES.
STREAM PATTERN = 2 STEMS.

TO FIND:

Q_{12}, T_{T12}, T_{R12} AND Q_{A12}.

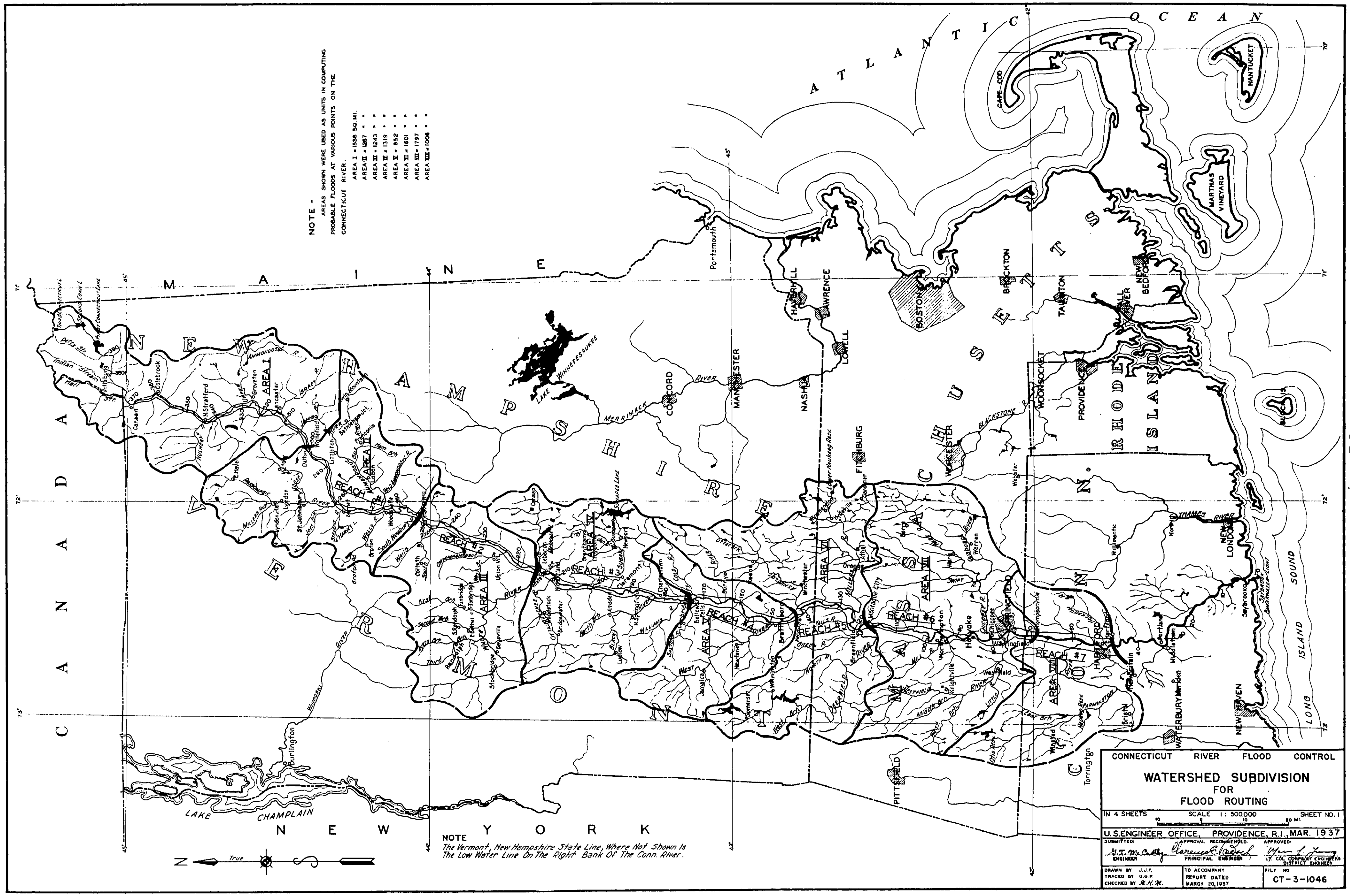
PROCEDURE:

ENTER CHART WITH S AS ILLUSTRATED AND OBTAIN FOLLOWING VALUES:
Q_{12} = 3200 C.F.S., T_{T12} = 4.9 DAYS, T_{R12} = 20 HOURS,
ENTER FIGURE 3 WITH 3200/200 AND OBTAIN Q_{A12} = 11.8 X 200 = 2360 C.F.S.
THE UNIT HYDROGRAPH CONSTRUCTED FROM THESE DATA IS SHOWN ON FIGURE 4.

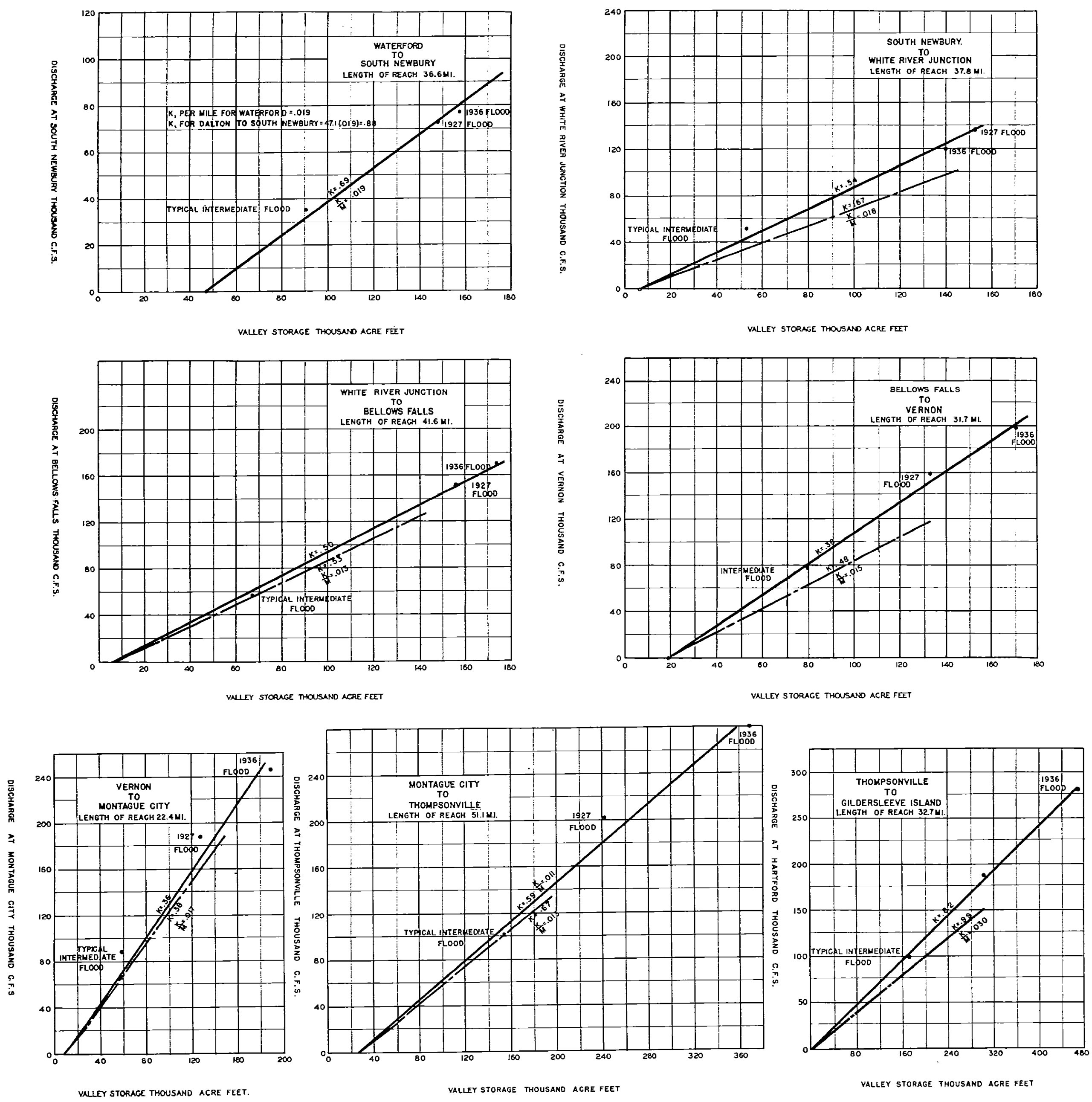
CONNECTICUT RIVER FLOOD CONTROL		
UNIT HYDROGRAPH RELATIONS		
IN 4 SHEETS	SCALE AS SHOWN	SHEET NO 4
U.S. ENGINEER OFFICE, PROVIDENCE, R.I.,		MAR. 1937
SUBMITTED: G. T. McCarthy ENGINEER	APPROVAL RECOMMENDED: Clarence ... PRINCIPAL ENGINEER	APPROVED: ... Young LT. COL. CORPS OF ENGINEERS DISTRICT ENGINEER
DRAWN BY L.P.F. & J.A.H. TRACED BY P.J.G. CHECKED BY D.V.M.	TO ACCOMPANY REPORT DATED MARCH 20, 1937	FILE NO CT-3-1057

WAR DEPARTMENT

CORPS OF ENGINEERS U.S.ARMY



SEC. I - PLATE NO. 16

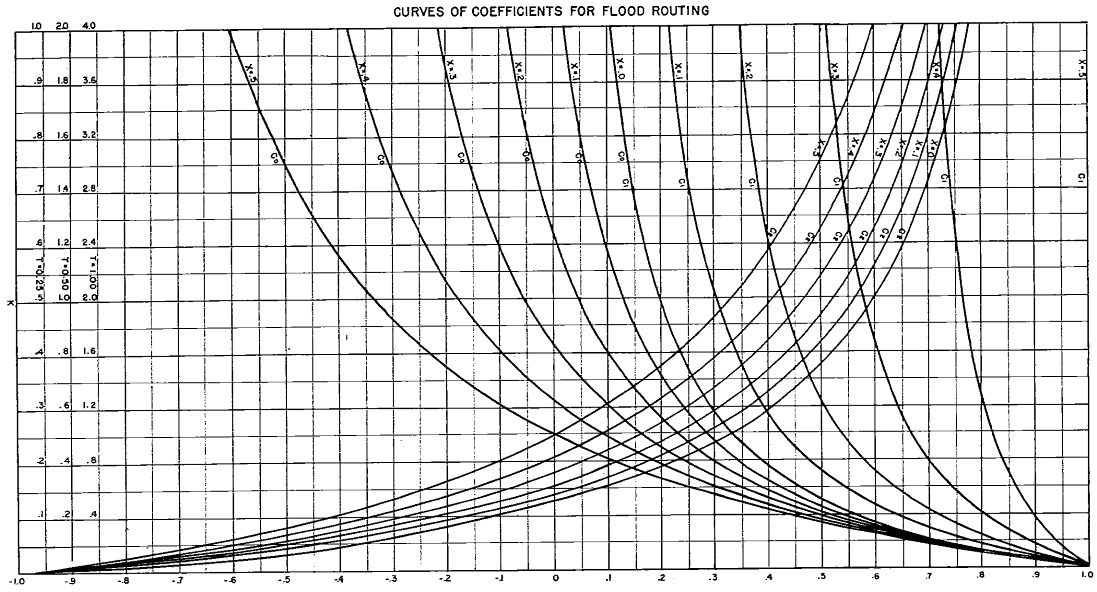


LEGEND

——— COMPUTED VALUE

— - — VALUE USED IN REPORT

CURVES OF COEFFICIENTS FOR FLOOD ROUTING



$d_2 = -\frac{Kx-.5T}{K-Kx+.5T} i_2 + \frac{Kx+.5T}{K-Kx+.5T} i_1 + \frac{K-Kx-.5T}{K-Kx+.5T} d_1 = C_0 i_2 + C_1 i_1 C_2 d_1$

i_1, i_2, ETC. = INFLOW RATE IN C.F.S. AT BEGINNING OF TIME UNITS

d_1, d_2, ETC. = OUTFLOW RATE IN C.F.S. AT BEGINNING OF TIME UNITS

$C_0, C_1, \& C_2$ = ROUTING COEFFICIENTS

T = TIME UNIT IN FRACTIONS OF A DAY

$K = \text{AVERAGE } \frac{\Delta \text{ VALLEY STORAGE}}{\Delta \text{ WEIGHTED FLOW}}$

X = PARAMETER OF WEIGHTED FLOW

CONNECTICUT RIVER FLOOD CONTROL

VALLEY STORAGE RELATIONS AND COEFFICIENTS OF FLOOD ROUTING

IN 4 SHEETS | SCALE AS SHOWN | SHEET NO. 2

U.S. ENGINEER OFFICE PROVIDENCE, R.I., MAR. 1937.

SUBMITTED: ENGINEER | APPROVAL RECOMMENDED: PRINCIPAL ENGINEER | APPROVED: LT. COL., CORPS OF ENGINEERS DISTRICT ENGINEER

DRAWN BY: I.G.L. TRACED BY: V.A.T. CHECKED BY:

TO ACCOMPANY REPORT DATED MARCH 22, 1937

FILE NO. CT-3-1059

CORPS OF ENGINEERS, U.S. ARMY.

WAR DEPARTMENT

TYPICAL RELATIONS OF DISCHARGE TO VALLEY STORAGE

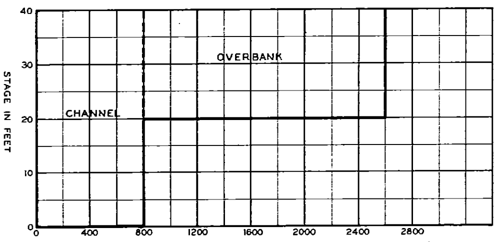


FIG. NO. 1

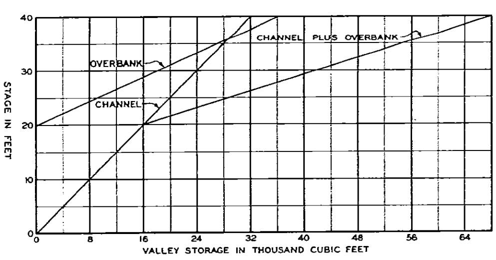


FIG. NO. 2

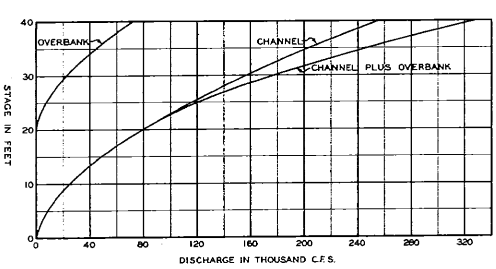


FIG. NO. 3

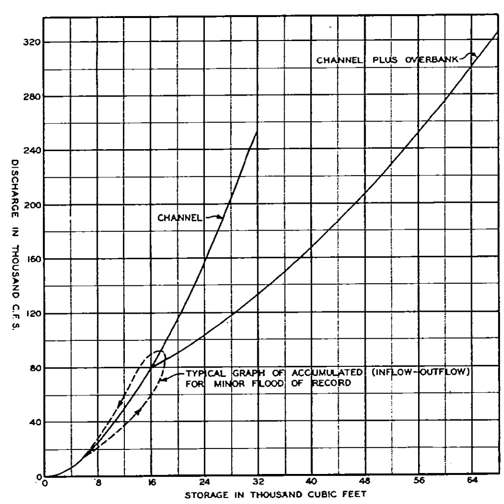
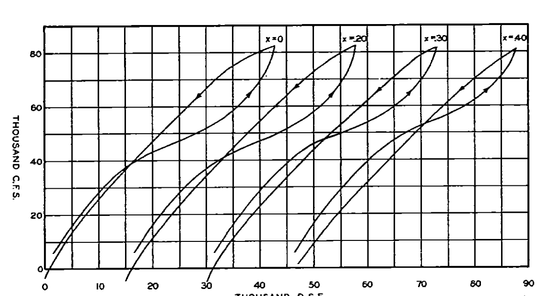


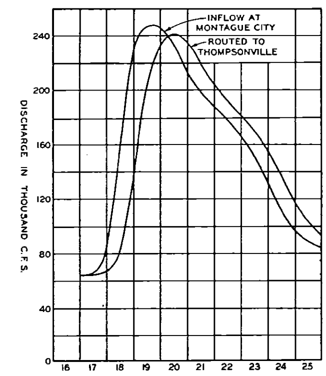
FIG. NO. 4



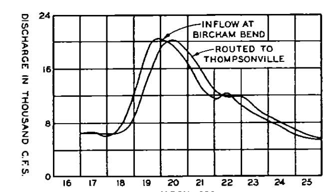
VALLEY STORAGE CURVES
FLOOD OF APRIL 15-25, 1933
MONTAGUE CITY-THOMPSONVILLE REACH
FIG. NO. 5

NOTES:
ORDINATES = $\Sigma\Delta$ WEIGHTED FLOW = $\Sigma[x(i_2-i_1)+(1.0-x)(d_2-d_1)]$
ABSCISSAE = $\Sigma\Delta$ VALLEY STORAGE = $\Sigma\{T[.5(i_2+i_1)-.5(d_2+d_1)]\}$
T = ½ DAY
x = PARAMETER OF WEIGHTED FLOW
i = INFLOW RATE IN C.F.S.
d = OUTFLOW RATE IN C.F.S.
$K = \dfrac{\Delta \text{VALLEY STORAGE}}{\Delta \text{WEIGHTED FLOW}}$

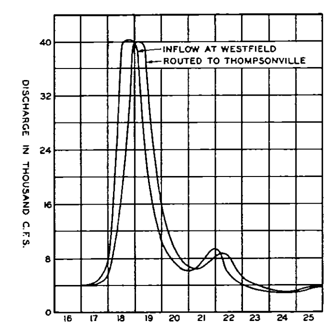
ROUTING OF MARCH 1936 FLOOD MONTAGUE CITY TO THOMPSONVILLE REACH



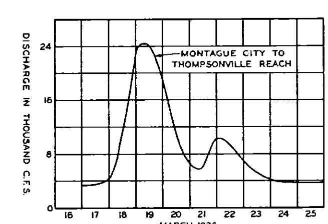
MARCH 1936
CONNECTICUT RIVER
D.A. = 7,840 SQ. MI.
FIG. NO. 6



MARCH 1936
CHICOPEE RIVER
D.A. = 703 SQ. MI.
FIG. NO. 7



MARCH 1936
WESTFIELD RIVER
D.A. = 497 SQ. MI.
FIG. NO. 8



MARCH 1936
LOCAL AT THOMPSONVILLE
D.A. = 297 SQ. MI.
FIG. NO. 9

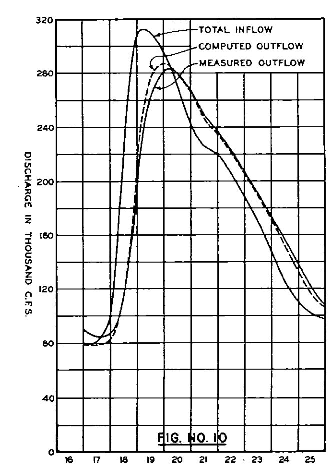
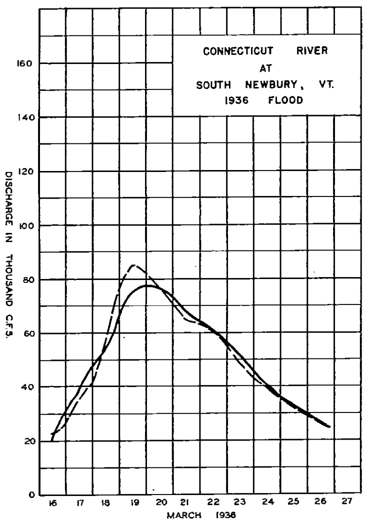
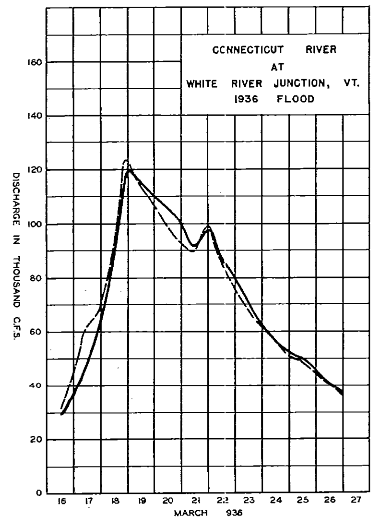


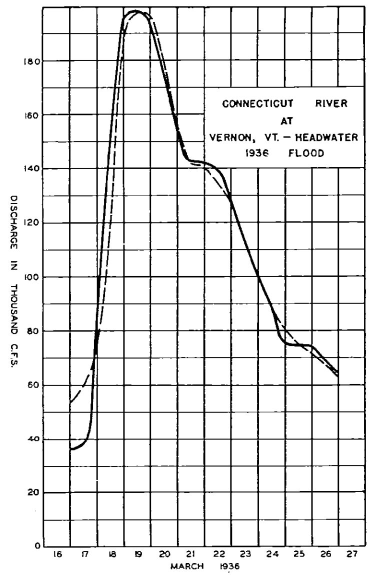
FIG. NO. 10
MARCH 1936
CONNECTICUT RIVER
AT
THOMPSONVILLE
D.A. = 9,637 SQ. MI.

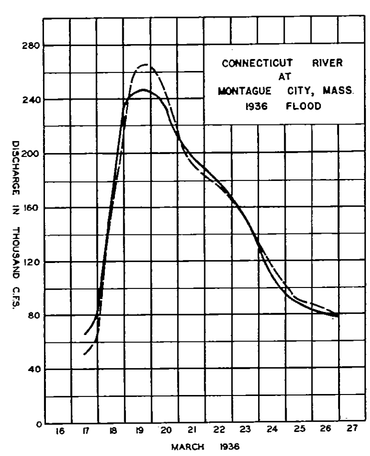
CONNECTICUT RIVER FLOOD CONTROL
TYPICAL VALLEY STORAGE RELATIONS
AND
FLOOD ROUTING
IN 4 SHEETS | SCALE AS SHOWN | SHEET NO. 3
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937
SUBMITTED | APPROVAL RECOMMENDED | APPROVED
ENGINEER | PRINCIPAL ENGINEER | DISTRICT ENGINEER
DRAWN BY W.G.L. TRACED BY W.L.H. CHECKED BY | TO ACCOMPANY REPORT DATED MARCH 20, 1937 | FILE NO. CT-3-1060

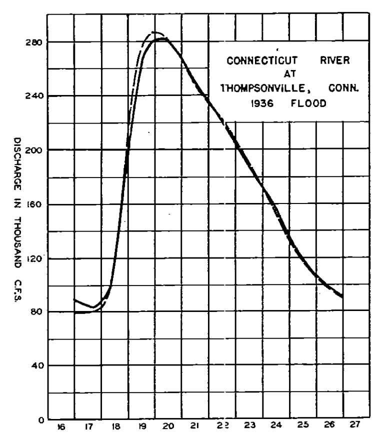
CORPS OF ENGINEERS, U.S. ARMY

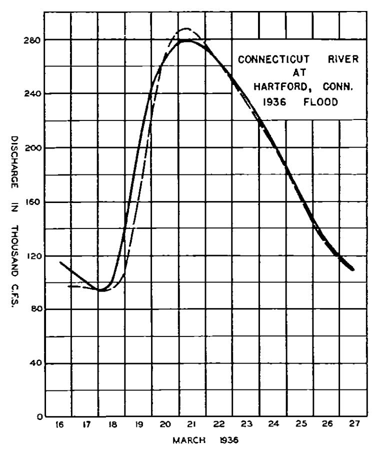








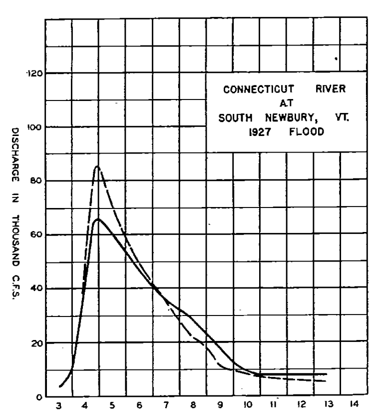


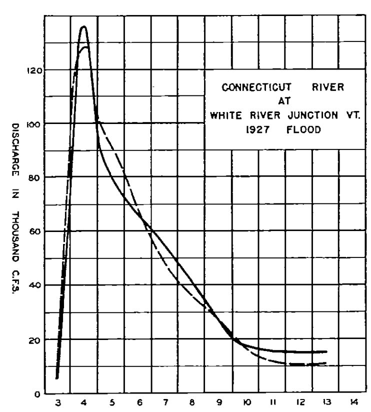


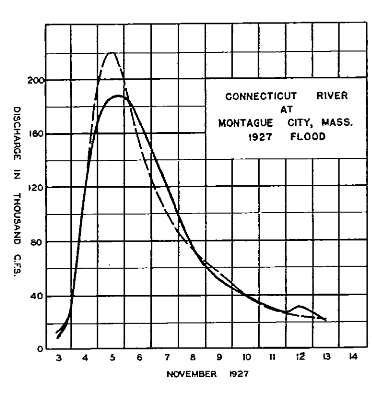
LEGEND

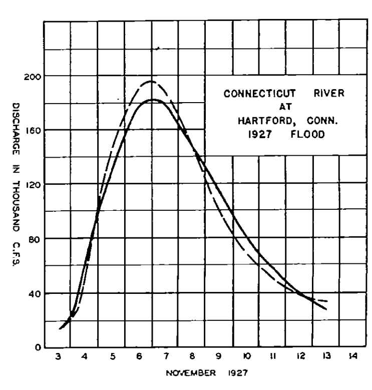
—— ACTUAL HYDROGRAPHS AS ESTABLISHED BY STREAM GAGES

– – COMPUTED HYDROGRAPHS AS ESTABLISHED BY FLOOD ROUTING









CONNECTICUT RIVER FLOOD CONTROL
COMPARISON OF
1927 AND 1936 FLOOD HYDROGRAPHS
FROM
RECORDS AND FLOOD ROUTING

IN 4 SHEETS — SCALE AS SHOWN — SHEET NO. 4

U.S. ENGINEER OFFICE, PROVIDENCE R.I., MAR. 1937

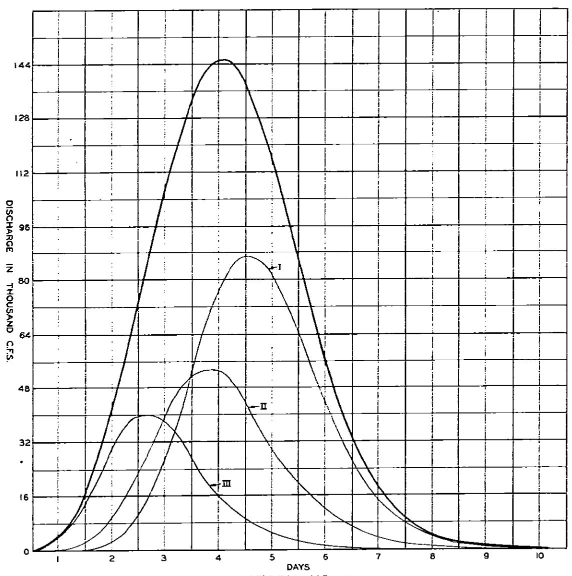
SUBMITTED: ENGINEER — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER — APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

TO ACCOMPANY REPORT DATED MARCH 20, 1937 — FILE NO. CT-3-1061

CORPS OF ENGINEERS, U. S. ARMY

SEC ... PLATE NO 16

WAR DEPARTMENT



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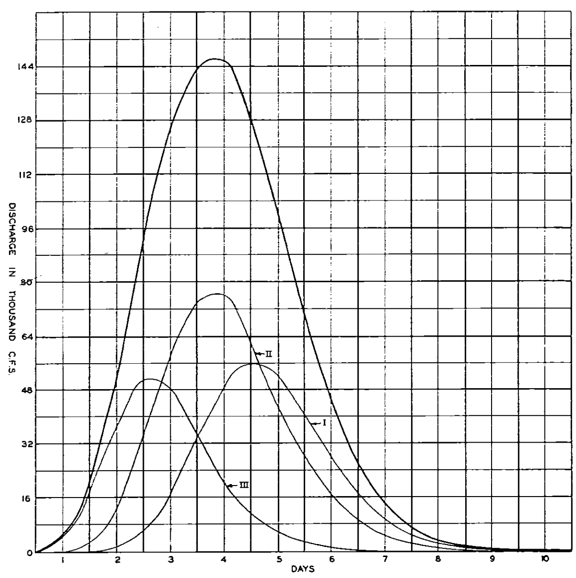
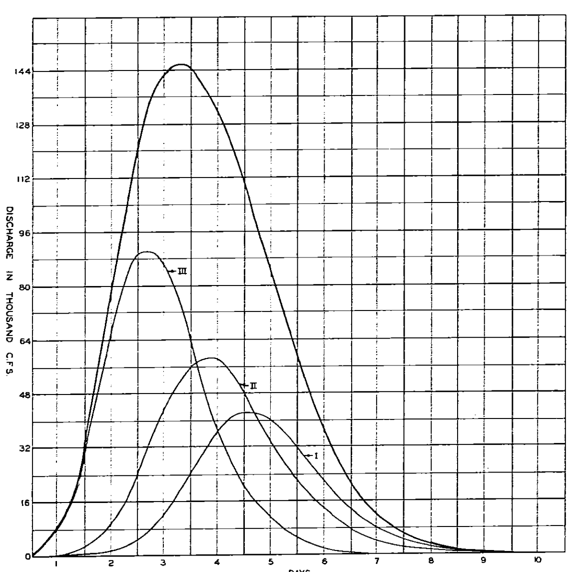
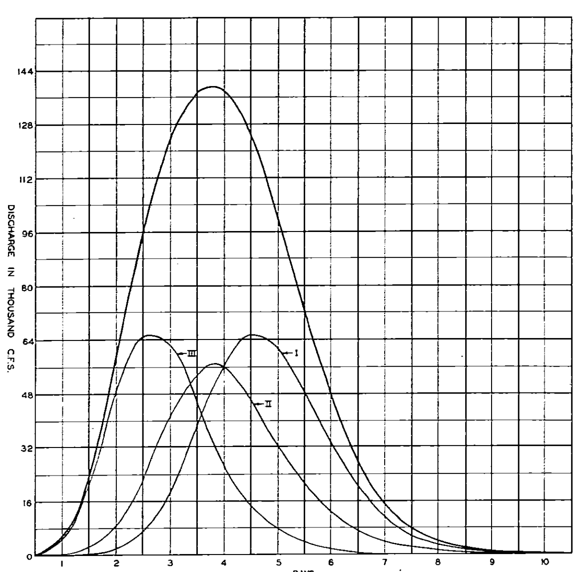


STORM NO. 2

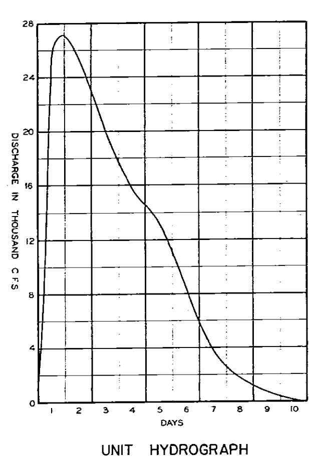


STORM NO. 3

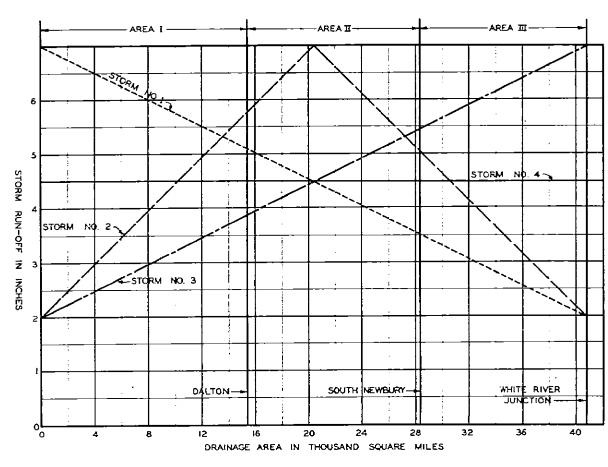


STORM NO. 4

NOTE: SEE DRAWING NO. CT-3-1046
FOR SUBDIVISION OF WATERSHED



UNIT HYDROGRAPH



DISTRIBUTION OF STORM RUN-OFF FOR 4.5 INCH FLOOD

CONNECTICUT RIVER FLOOD CONTROL

PROBABLE FLOODS

WITH VARIOUS DISTRIBUTIONS OF RAINFALL

AT

WHITE RIVER JUNCTION VERMONT

IN 6 SHEETS SCALE AS SHOWN SHEET NO. 1

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

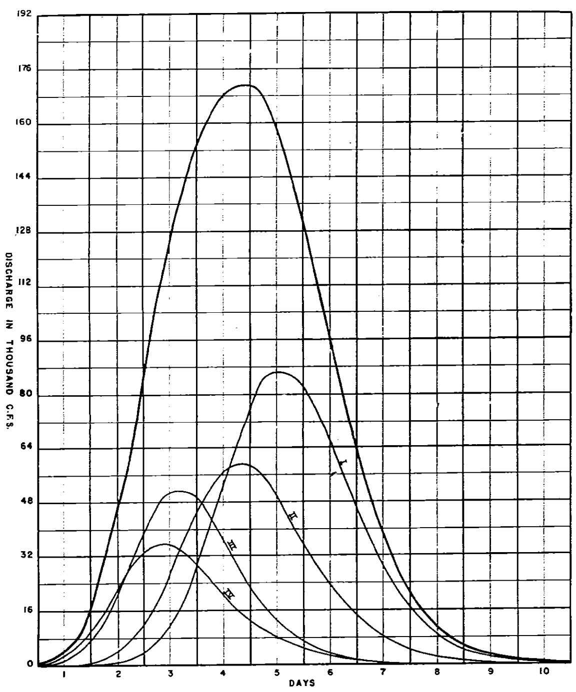
SUBMITTED: B. T. McCarthy ENGINEER
APPROVAL RECOMMENDED: Clarence C. Beach PRINCIPAL ENGINEER
APPROVED: Wm. F. Tompkins LT. COL., CORPS OF ENGINEERS DISTRICT ENGINEER

DRAWN BY J.E.H.
TRACED BY J.E.P. & L.W.B.
CHECKED BY B. H. M.

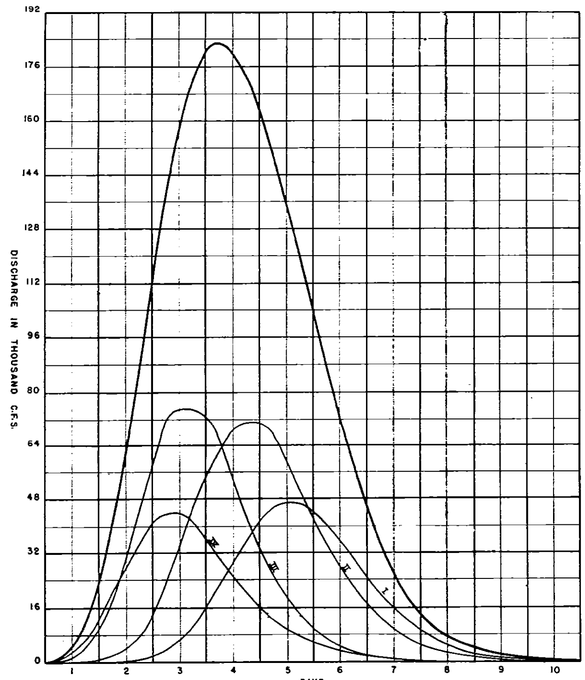
TO ACCOMPANY REPORT DATED MARCH 20, 1937

FILE NO. CT-3-1067

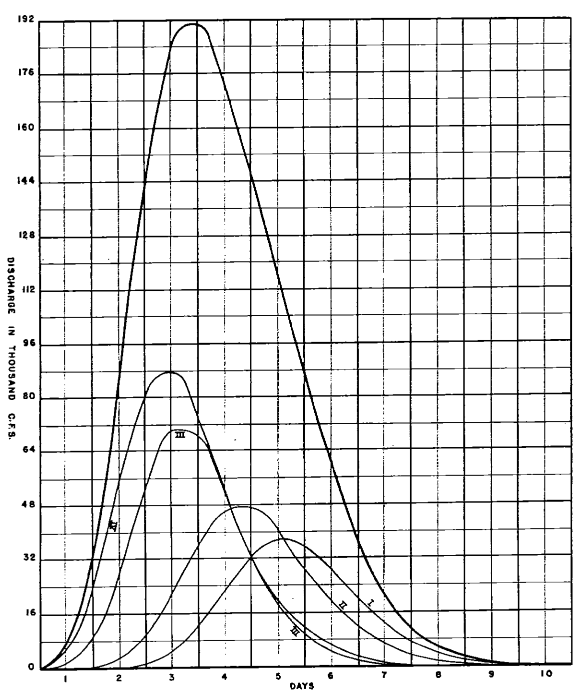
CORPS OF ENGINEERS U.S. ARMY



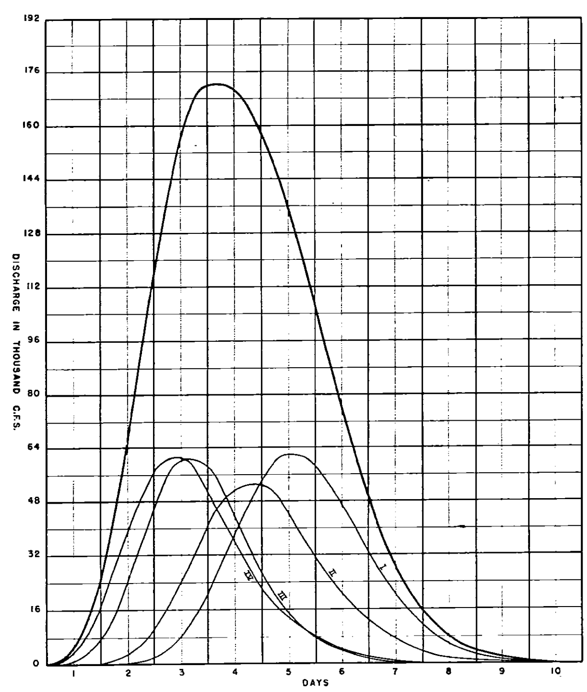
STORM NO. 1



STORM NO. 2

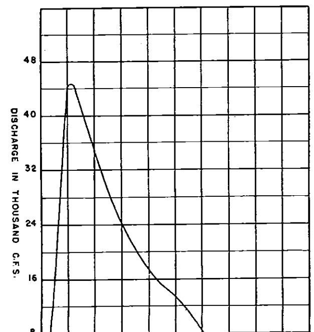


STORM NO. 3

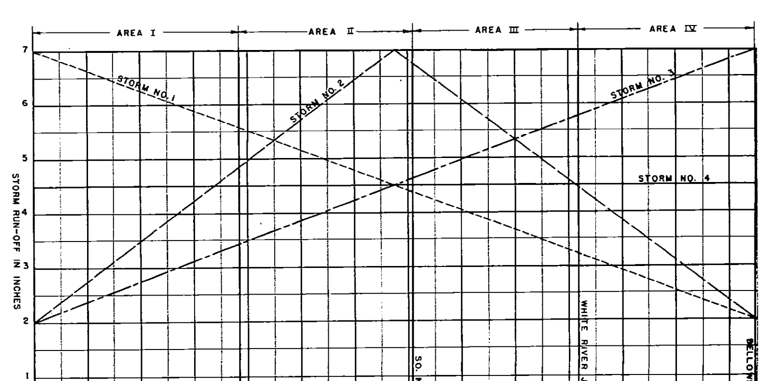


STORM NO. 4

NOTE: SEE DWG. NO. CT-3-1046 FOR SUBDIVISION OF WATERSHED



UNIT HYDROGRAPH



DISTRIBUTION OF STORM RUN-OFF FOR 4.5 INCH FLOOD

CONNECTICUT RIVER FLOOD CONTROL

PROBABLE FLOODS

WITH VARIOUS DISTRIBUTIONS OF RAINFALL

AT

BELLOWS FALLS VERMONT

IN 6 SHEETS | SCALES AS SHOWN | SHEET NO. 2

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR. 1937

SUBMITTED: | APPROVAL RECOMMENDED: | APPROVED:

ENGINEER | PRINCIPAL ENGINEER | LT. COL. CORPS OF ENGINEERS DISTRICT ENGINEER

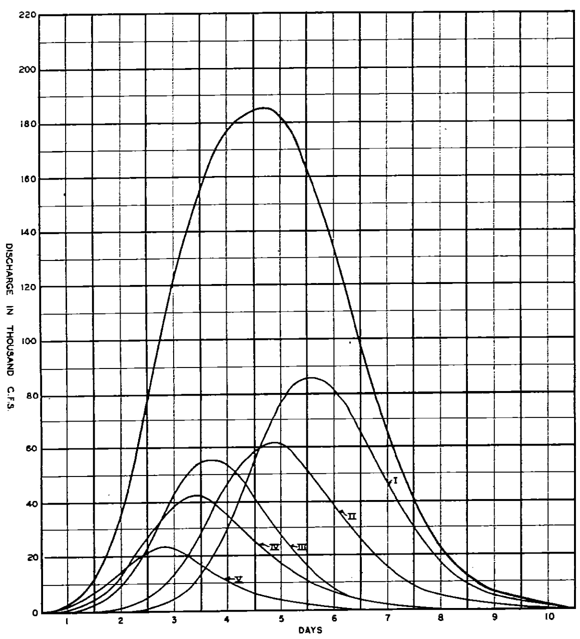
DRAWN BY: J.E.H. TRACED BY: J.F.J. CHECKED BY:

TO ACCOMPANY REPORT DATED MARCH 20, 1937

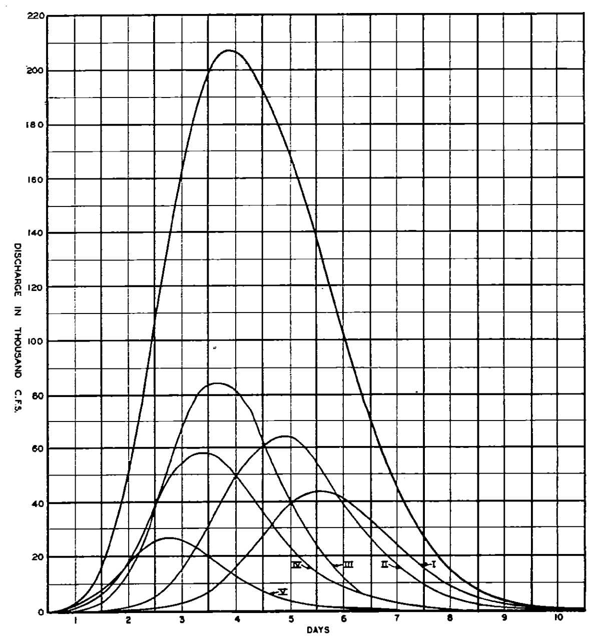
FILE NO. CT-3-1066

CORPS OF ENGINEERS, U.S. ARMY

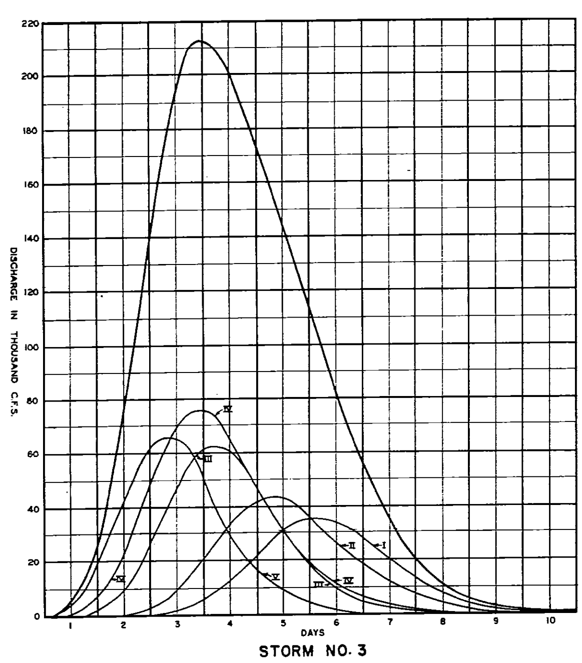
WAR DEPARTMENT



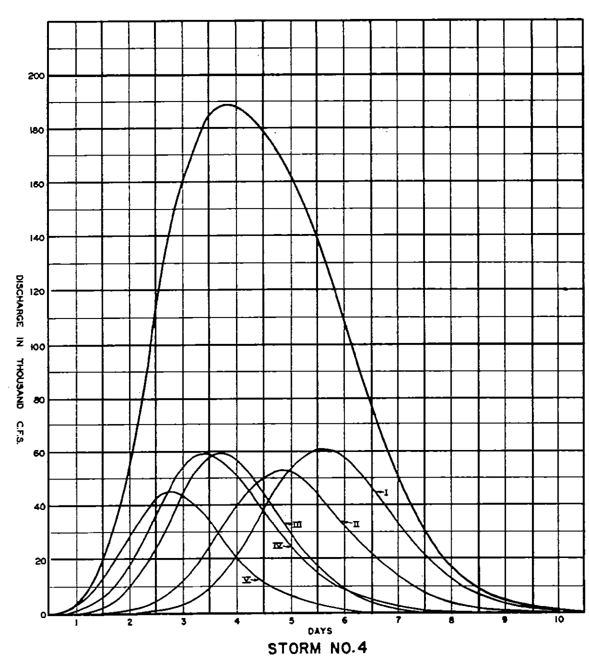
STORM NO. 1



STORM NO. 2

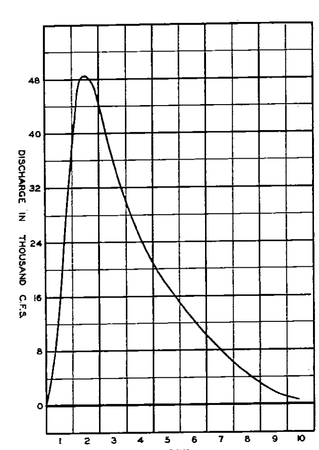


STORM NO. 3

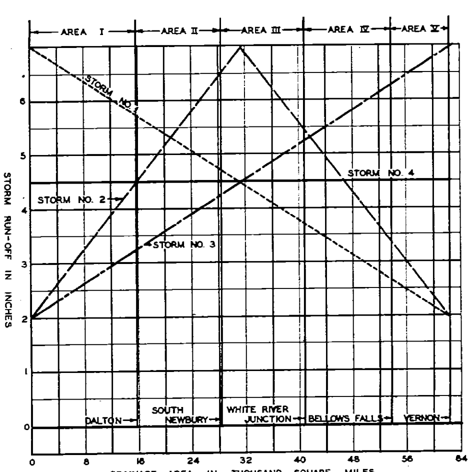


STORM NO. 4

NOTE:
SEE DWG. NO. CT-3-1046 FOR SUBDIVISION OF WATERSHED



UNIT HYDROGRAPH



DISTRIBUTION OF STORM RUN-OFF FOR 4.5" FLOOD

CONNECTICUT RIVER FLOOD CONTROL

PROBABLE FLOODS

WITH VARIOUS DISTRIBUTIONS OF RAINFALL

AT

VERNON VERMONT

IN 6 SHEETS SCALE AS SHOWN SHEET NO. 3

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: ENGINEER | APPROVAL RECOMMENDED: PRINCIPAL ENGINEER | APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

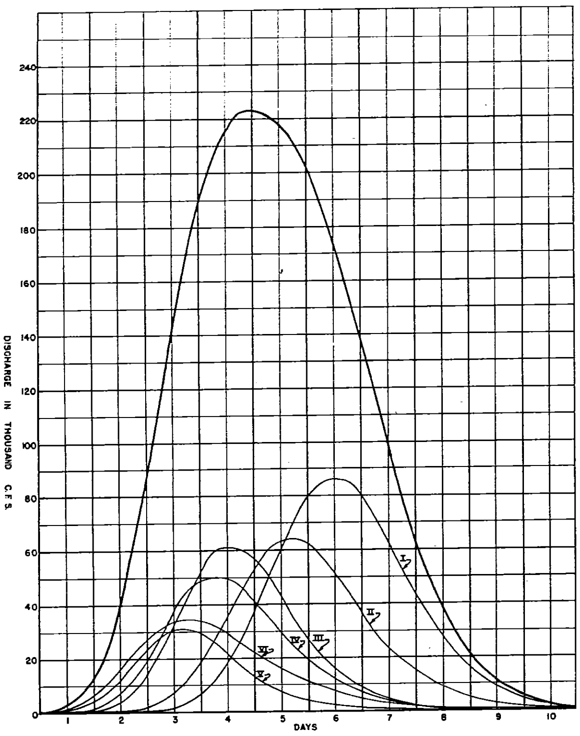
DRAWN BY: J.E.H.
TRACED BY: L.W.B.
CHECKED BY:

TO ACCOMPANY REPORT DATED MARCH 20, 1937

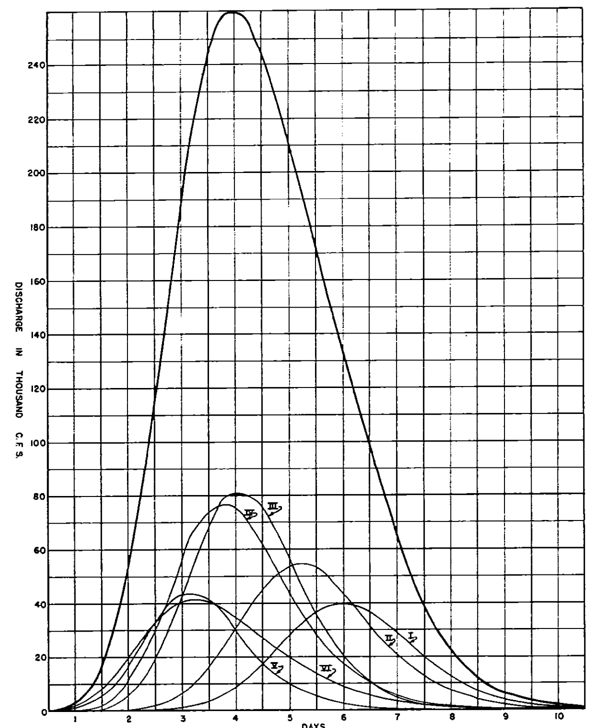
FILE NO. CT-3-1065

CORPS OF ENGINEERS, U.S. ARMY

WAR DEPARTMENT



STORM NO. 1



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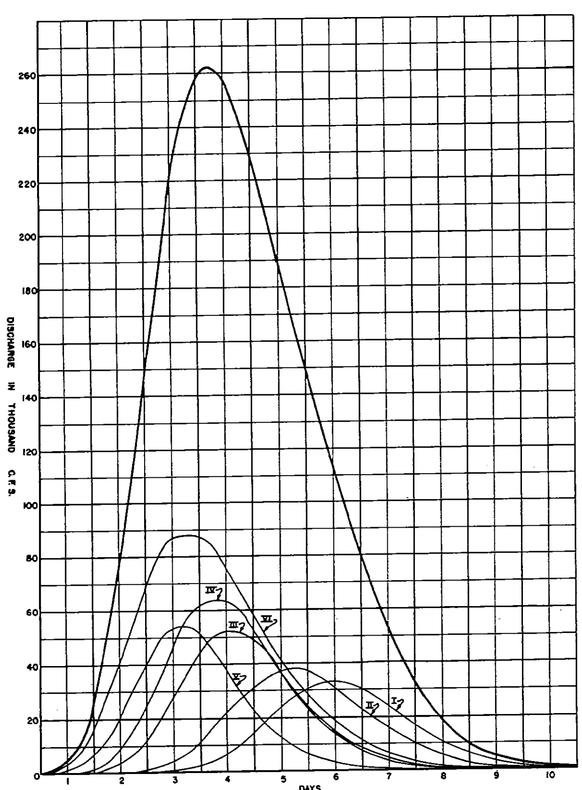
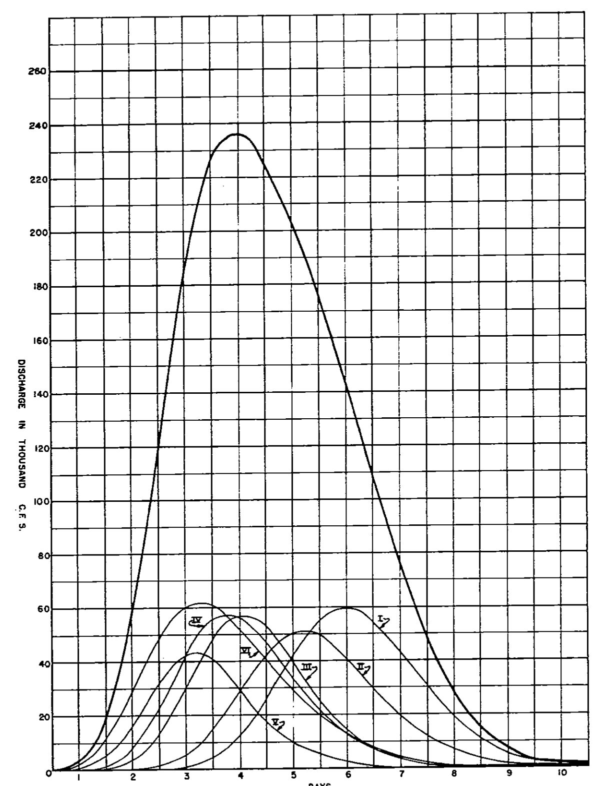
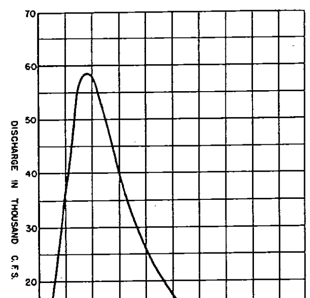


STORM NO. 3

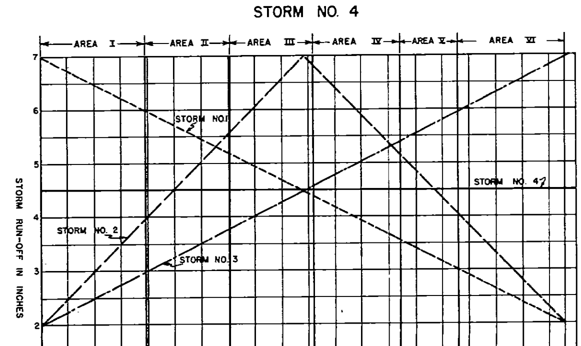
NOTE - SEE DRAWING NO. CT-3-1046 FOR SUBDIVISIONS OF WATERSHED.



STORM NO. 4



UNIT HYDROGRAPH



DISTRIBUTION OF STORM RUN-OFF FOR 4.5 INCH FLOOD

CONNECTICUT RIVER FLOOD CONTROL

PROBABLE FLOODS

WITH VARIOUS DISTRIBUTIONS OF RAINFALL

AT

MONTAGUE CITY, MASSACHUSETTS

IN 6 SHEETS | SCALE AS SHOWN | SHEET NO. 4

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: ENGINEER | APPROVAL RECOMMENDED: PRINCIPAL ENGINEER | APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

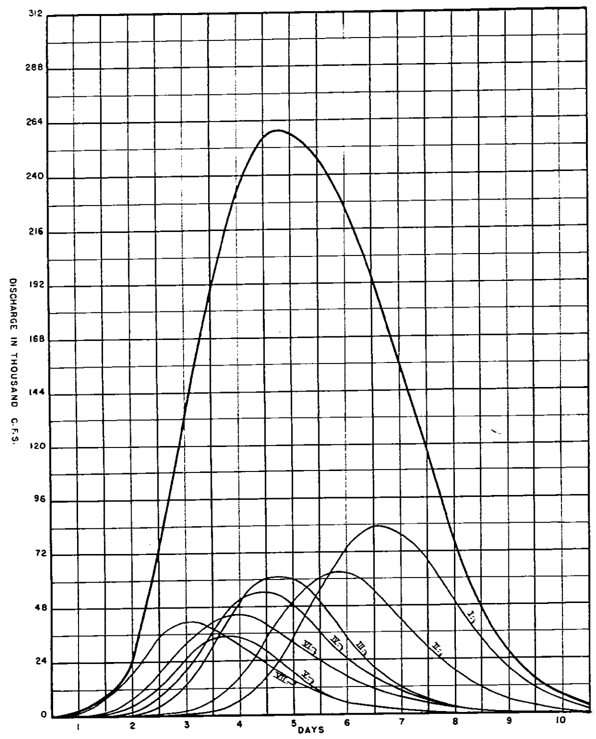
DRAWN BY J.E.M.
TRACED BY E.G.
CHECKED BY

TO ACCOMPANY REPORT DATED MARCH 20, 1937

FILE NO. CT-3-1064

CORPS OF ENGINEERS, U. S. ARMY

WAR DEPARTMENT



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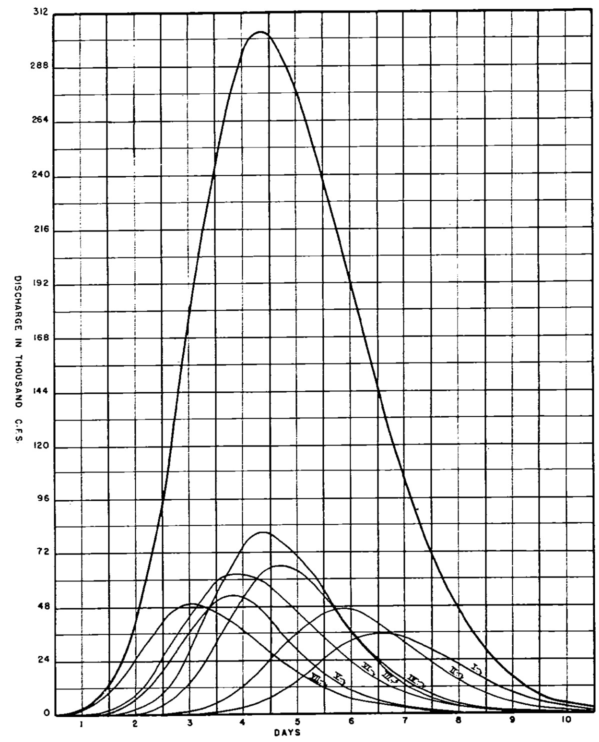
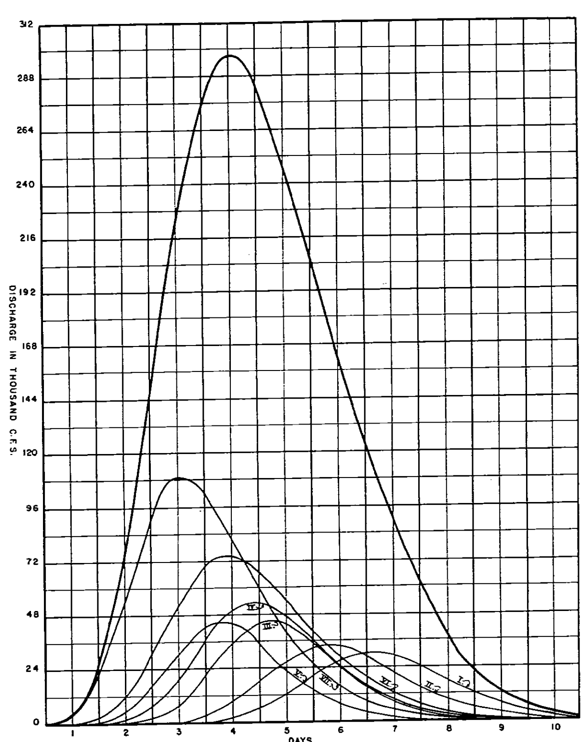
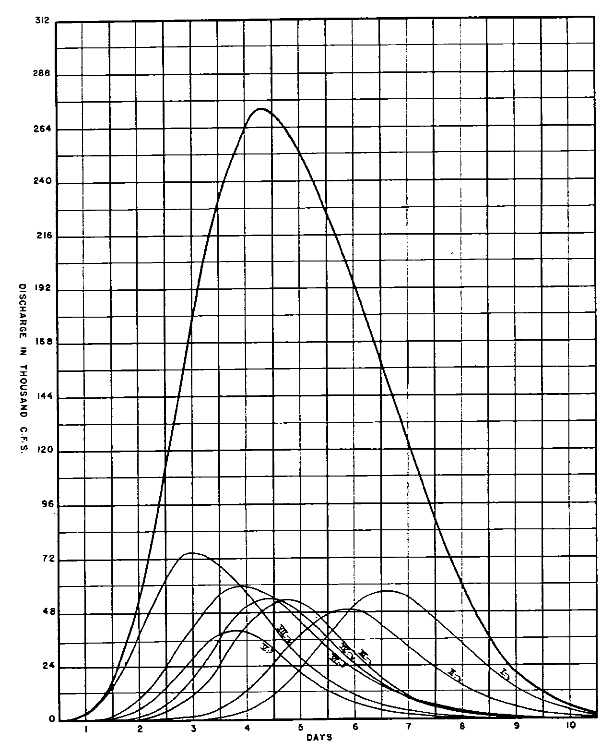


STORM NO. 2

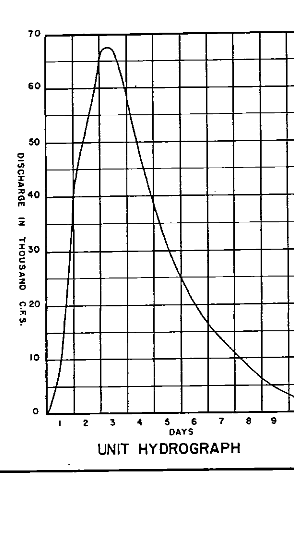


STORM NO. 3



STORM NO. 4

NOTE:
SEE DWG. NO. CT-3-1046 FOR SUBDIVISION OF WATERSHED



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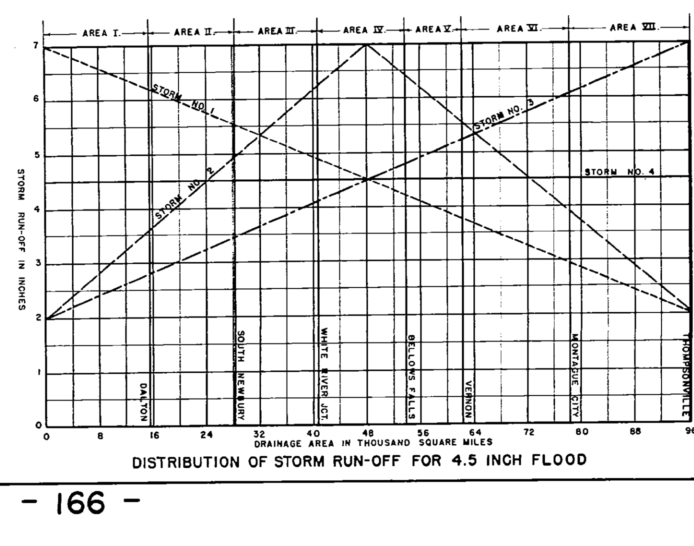


DISTRIBUTION OF STORM RUN-OFF FOR 4.5 INCH FLOOD

CONNECTICUT RIVER FLOOD CONTROL

PROBABLE FLOODS
WITH VARIOUS DISTRIBUTIONS OF RAINFALL
AT
THOMPSONVILLE CONNECTICUT

IN 6 SHEETS | SCALES AS SHOWN | SHEET NO. 5

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR. 1937

SUBMITTED: ENGINEER
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER
APPROVED: LT. COL. CORPS OF ENGINEERS DISTRICT ENGINEER

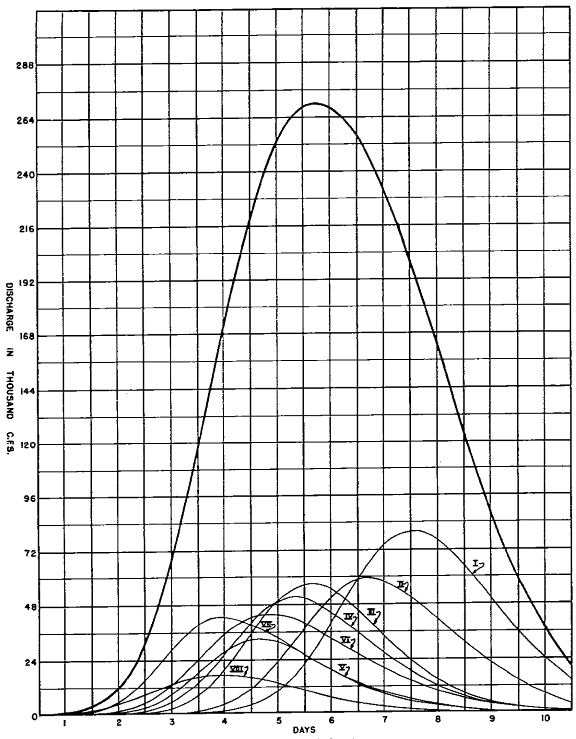
DRAWN BY: J.E.M.
TRACED BY: J.F.J.
CHECKED BY:

TO ACCOMPANY REPORT DATED MARCH 10, 1937

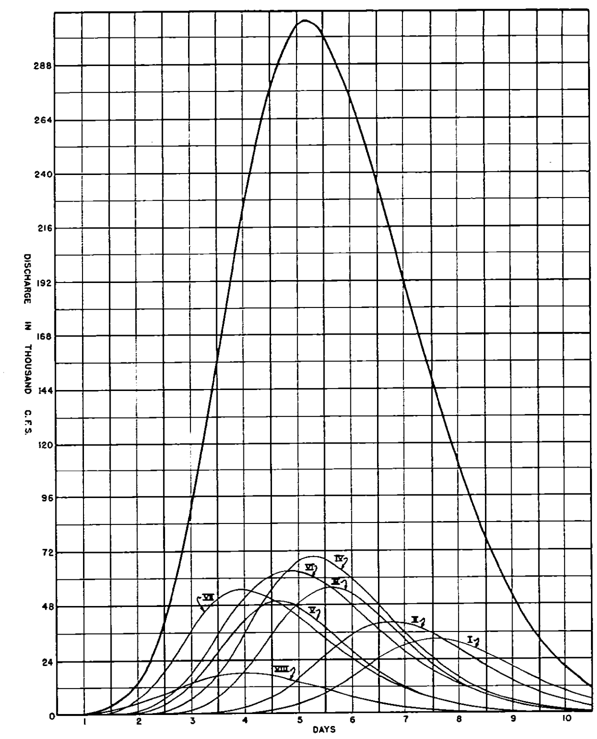
FILE NO. CT-3-1063

CORPS OF ENGINEERS U.S. ARMY

WAR DEPARTMENT



STORM NO. 1



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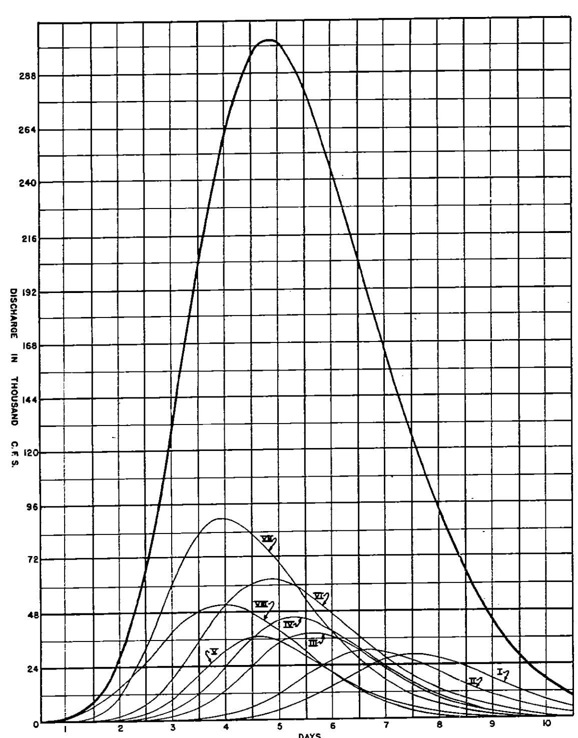

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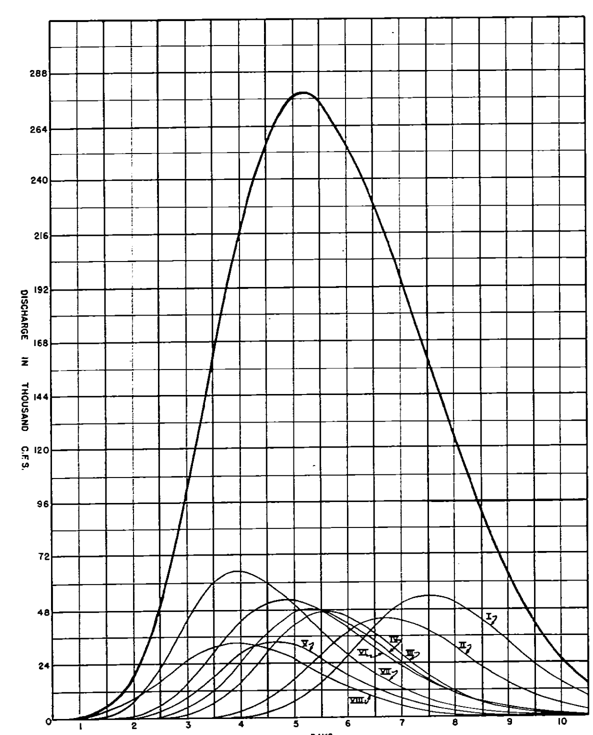
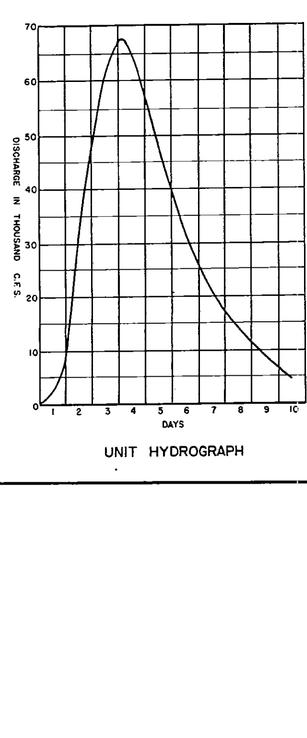
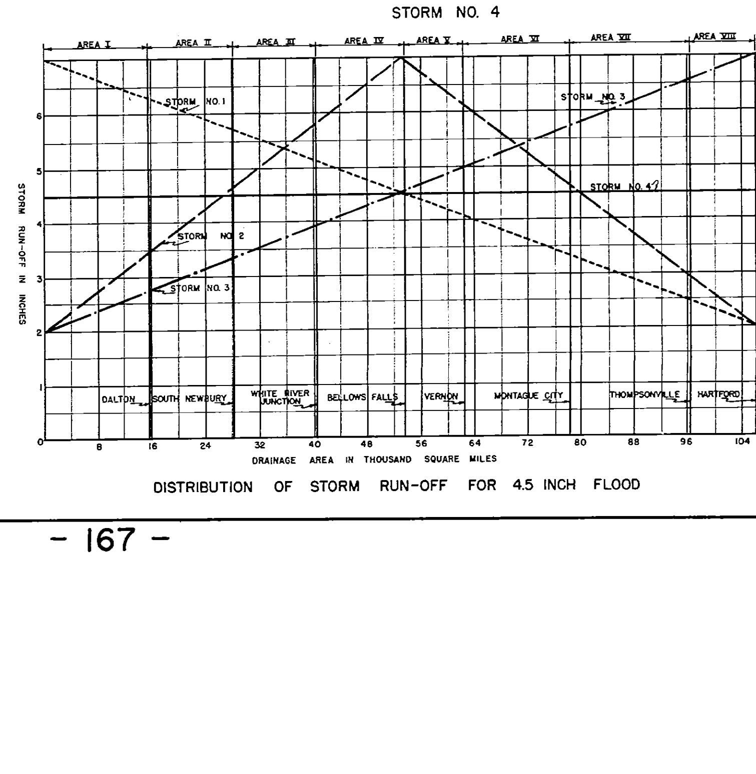


STORM NO. 4

NOTE:
SEE DWG. NO. CT-3-1046 FOR SUBDIVISION OF WATERSHED.



UNIT HYDROGRAPH



DISTRIBUTION OF STORM RUN-OFF FOR 4.5 INCH FLOOD

CONNECTICUT RIVER FLOOD CONTROL
PROBABLE FLOODS
WITH VARIOUS DISTRIBUTIONS OF RAINFALL
AT
HARTFORD, CONNECTICUT

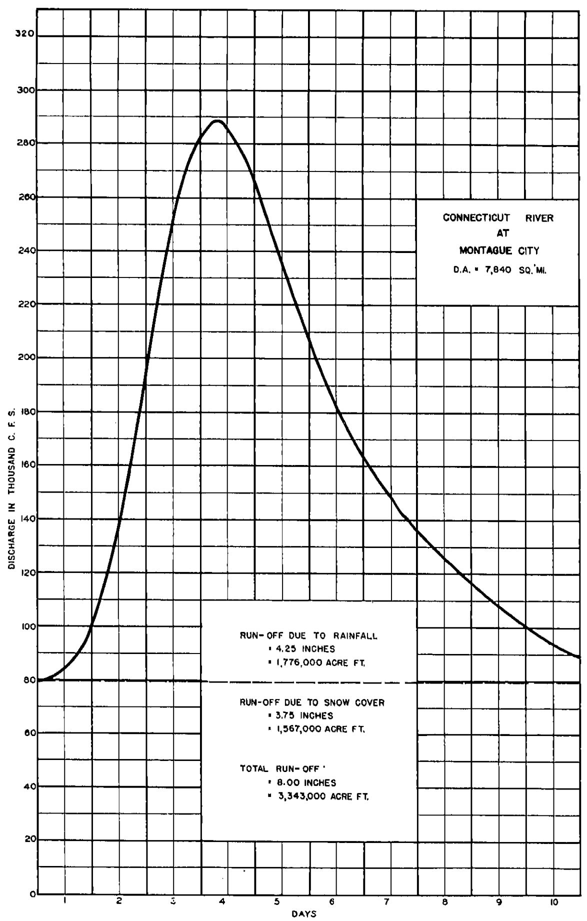
IN 6 SHEETS | SCALE AS SHOWN | SHEET NO. 6

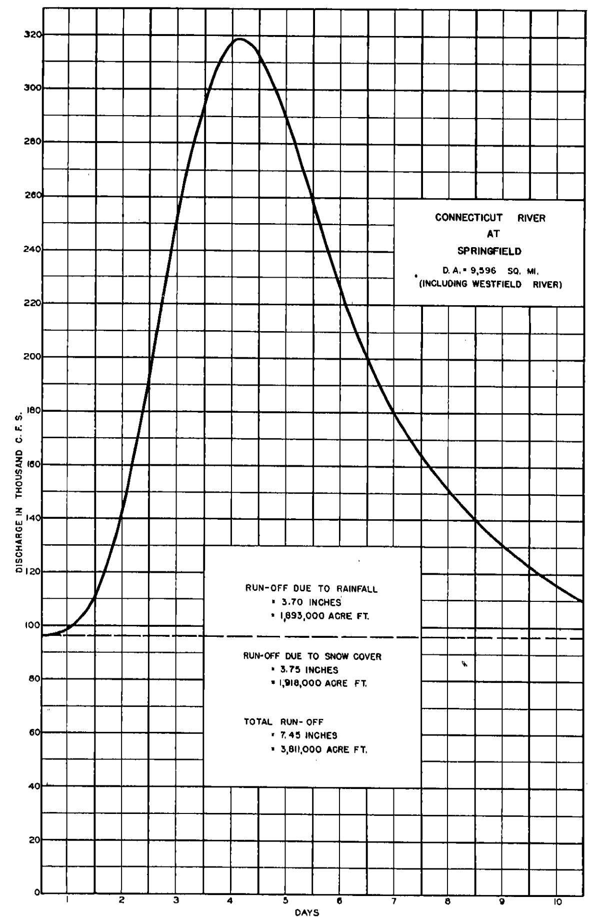
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

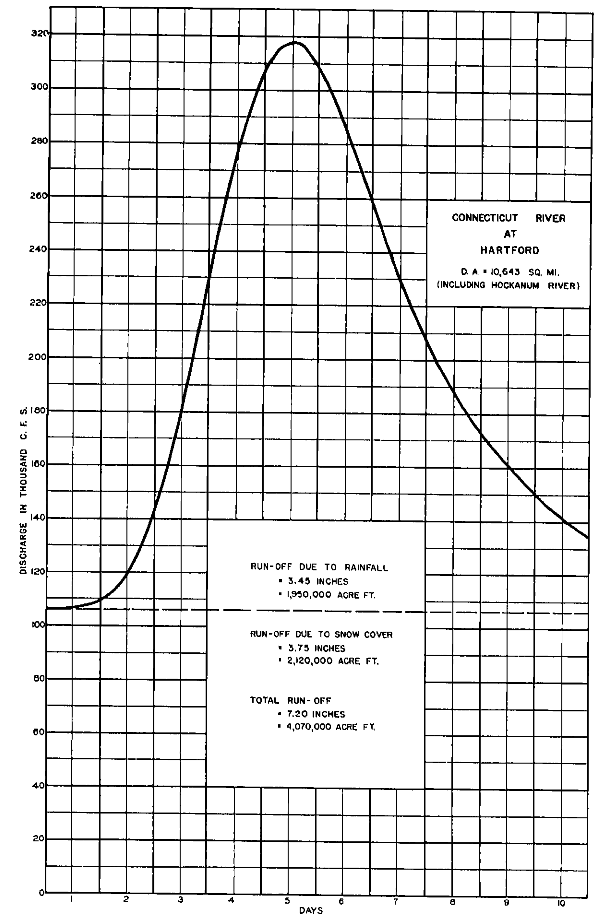
SUBMITTED: ENGINEER | APPROVAL RECOMMENDED: PRINCIPAL ENGINEER | APPROVED: CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY W.G.M. TRACED BY J.E.P. CHECKED BY | TO ACCOMPANY REPORT DATED MARCH 1937 | FILE NO. CT-3-1062

CORPS OF ENGINEERS, U. S. ARMY







CONNECTICUT RIVER FLOOD CONTROL

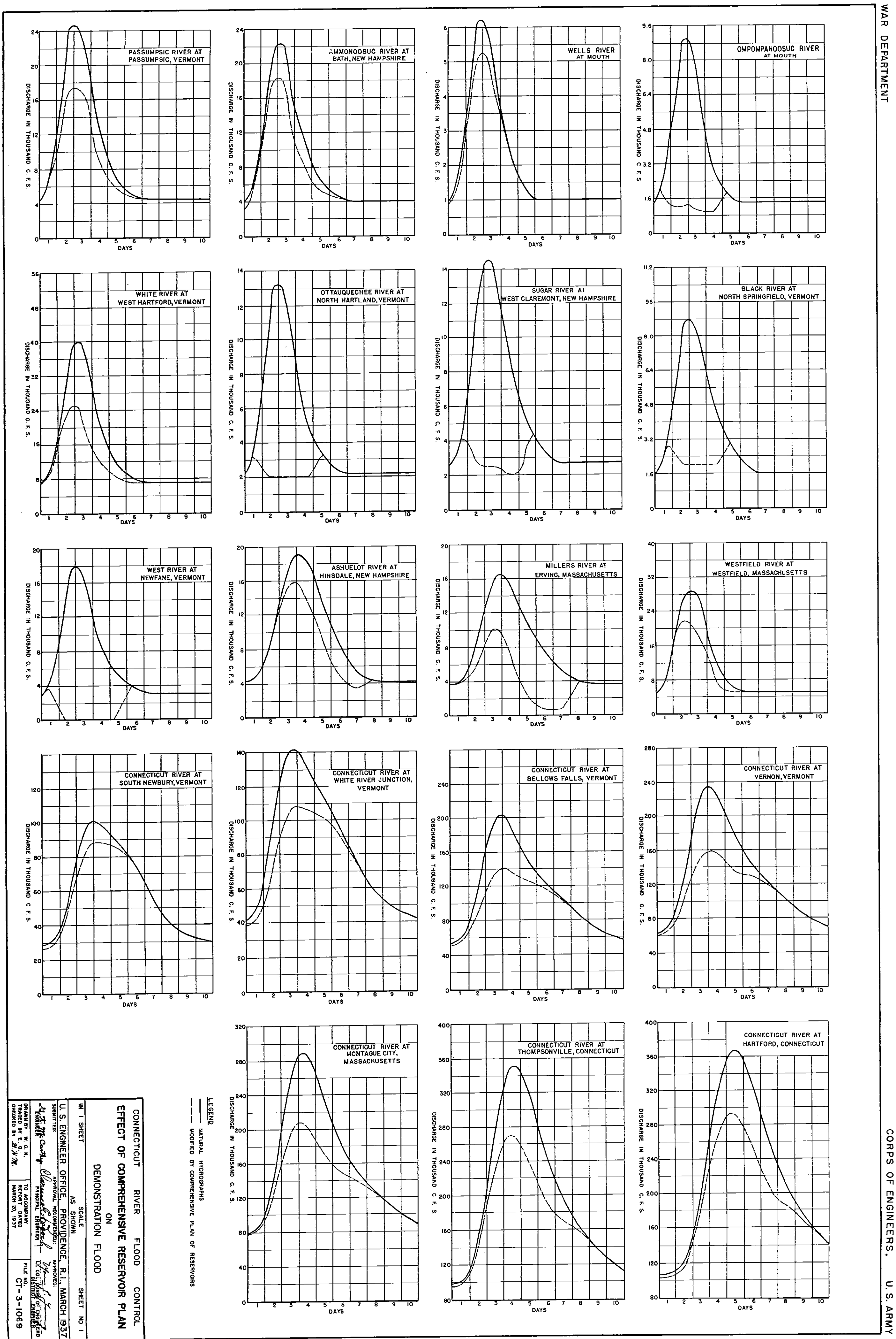
MAXIMUM PREDICTED FLOODS AT MONTAGUE CITY, SPRINGFIELD, AND HARTFORD

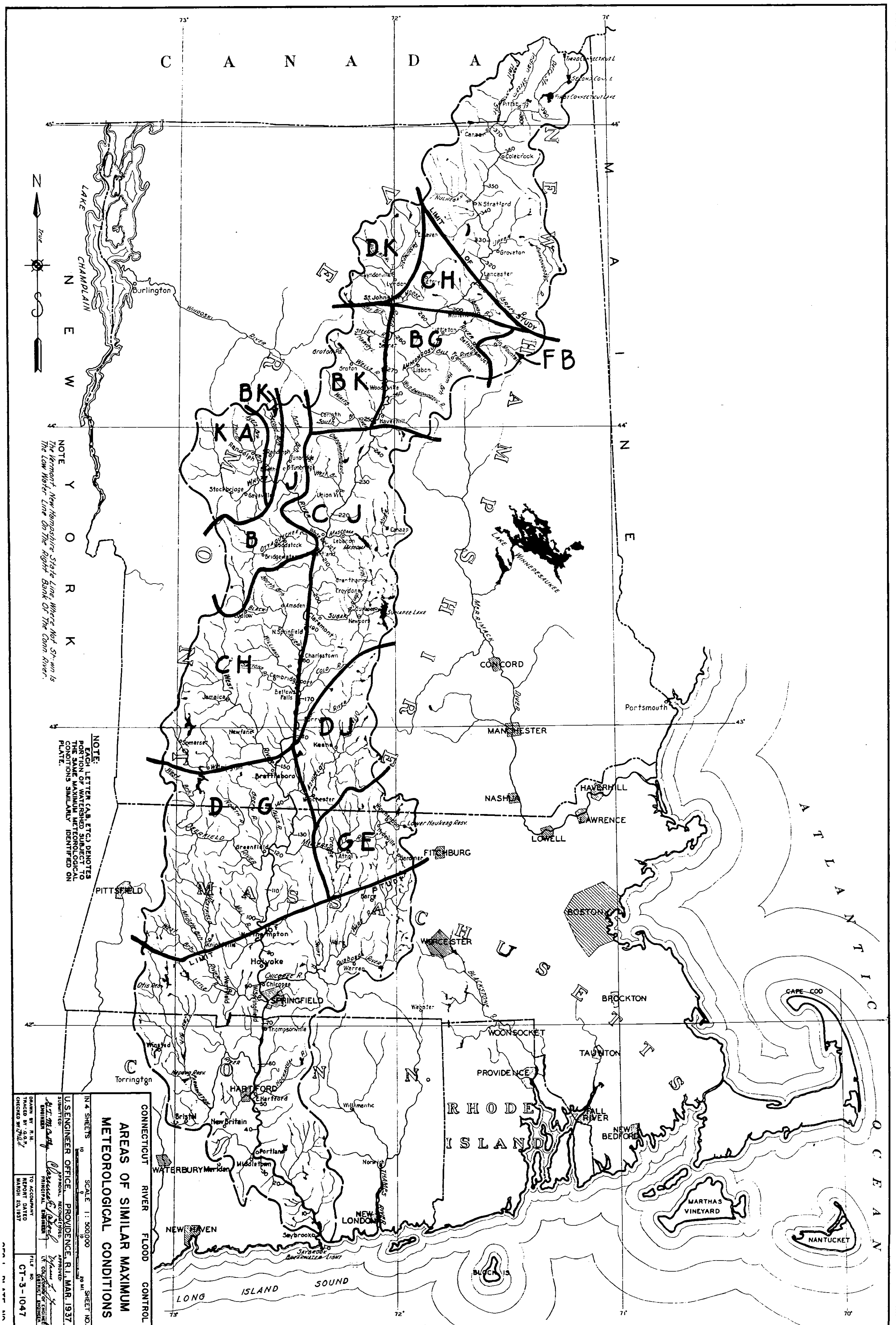
IN 1 SHEET — SCALE AS SHOWN — SHEET NO 1

U. S. ENGINEER OFFICE PROVIDENCE, R. I., MAR. 1937

SUBMITTED: G. T. McCarthy, ENGINEER — APPROVAL RECOMMENDED: Clarence [illegible], PRINCIPAL ENGINEER — APPROVED: [illegible], LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY W. C. N. TRACED BY H. O. B. H. C. W. CHECKED BY G. H. K. — TO ACCOMPANY REPORT DATED MARCH 20, 1937 — FILE NO. CT-3-1068


WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY
PASSUMPSIC RIVER AT PASSUMPSIC, VERMONT
AMMONOOSUC RIVER AT BATH, NEW HAMPSHIRE
WELLS RIVER AT MOUTH
OMPOMPANOOSUC RIVER AT MOUTH
WHITE RIVER AT WEST HARTFORD, VERMONT
OTTAUQUECHEE RIVER AT NORTH HARTLAND, VERMONT
SUGAR RIVER AT WEST CLAREMONT, NEW HAMPSHIRE
BLACK RIVER AT NORTH SPRINGFIELD, VERMONT
WEST RIVER AT NEWFANE, VERMONT
ASHUELOT RIVER AT HINSDALE, NEW HAMPSHIRE
MILLERS RIVER AT ERVING, MASSACHUSETTS
WESTFIELD RIVER AT WESTFIELD, MASSACHUSETTS
CONNECTICUT RIVER AT SOUTH NEWBURY, VERMONT
CONNECTICUT RIVER AT WHITE RIVER JUNCTION, VERMONT
CONNECTICUT RIVER AT BELLOWS FALLS, VERMONT
CONNECTICUT RIVER AT VERNON, VERMONT
CONNECTICUT RIVER AT MONTAGUE CITY, MASSACHUSETTS
CONNECTICUT RIVER AT THOMPSONVILLE, CONNECTICUT
CONNECTICUT RIVER AT HARTFORD, CONNECTICUT
DISCHARGE IN THOUSAND C. F. S.
DAYS
LEGEND
NATURAL HYDROGRAPHS
MODIFIED BY COMPREHENSIVE PLAN OF RESERVOIRS
CONNECTICUT RIVER FLOOD CONTROL
EFFECT OF COMPREHENSIVE RESERVOIR PLAN
ON
DEMONSTRATION FLOOD
IN 1 SHEET
SCALE AS SHOWN
SHEET NO 1
U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1937
SUBMITTED:
ENGINEER
APPROVAL RECOMMENDED:
PRINCIPAL ENGINEER
APPROVED:
LT. COL., CORPS OF ENGINEERS
DISTRICT ENGINEER
DRAWN BY W. G. N.
TRACED BY E. G.
CHECKED BY
TO ACCOMPANY REPORT DATED MARCH 20, 1937
FILE NO. CT-3-1069


WAR DEPARTMENT
CORPS OF ENGINEERS U.S.ARMY
CANADA
NEW YORK
VERMONT
NEW HAMPSHIRE
MAINE
MASSACHUSETTS
RHODE ISLAND
CONN.
ATLANTIC OCEAN
LONG ISLAND SOUND
LAKE CHAMPLAIN
LAKE WINNEPESAUKEE
DK
CH
BG
FB
BK
KA
J
CJ
B
DJ
DG
GE
LIMIT OF STUDY
Burlington
Concord
Manchester
Portsmouth
Haverhill
Lawrence
Lowell
Nashua
Fitchburg
Boston
Worcester
Brockton
Taunton
Woonsocket
Providence
Fall River
New Bedford
Cape Cod
Martha's Vineyard
Nantucket
Block Is.
Pittsfield
Springfield
Holyoke
Northampton
Hartford
New Britain
Middletown
Waterbury
Meriden
New Haven
New London
Saybrook
Torrington
Keene
Brattleboro
Bellows Falls
Claremont
Hanover
Woodsville
Lancaster
Colebrook
NOTE: EACH LETTER (A,B, ETC.) DENOTES PORTION OF WATERSHED SUBJECT TO THE SAME MAXIMUM METEOROLOGICAL CONDITIONS SIMILARLY IDENTIFIED ON PLATE.
NOTE The Vermont, New Hampshire State Line, Where Not Shown Is The Low Water Line On The Right Bank Of The Conn. River.
CONNECTICUT RIVER FLOOD CONTROL
AREAS OF SIMILAR MAXIMUM METEOROLOGICAL CONDITIONS
IN 4 SHEETS SCALE 1:500000 SHEET NO. 1
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR. 1937
CT-3-1047

WAR DEPARTMENT

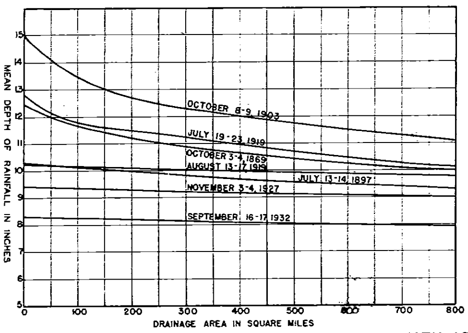


FIG. (1) GRAPHS OF DRAINAGE AREA VS. MEAN DEPTH OF RAINFALL FOR THE GREAT SUMMER AND FALL STORMS OF RECORD IN NORTHEASTERN UNITED STATES

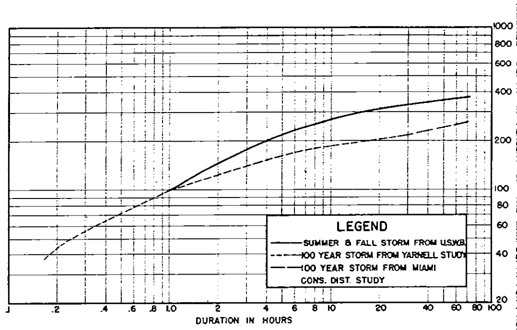


FIG. (5) DURATION VS. RAINFALL DEPTH FROM THE U. S. WEATHER BUREAU STUDY AND STUDIES OF RECORD

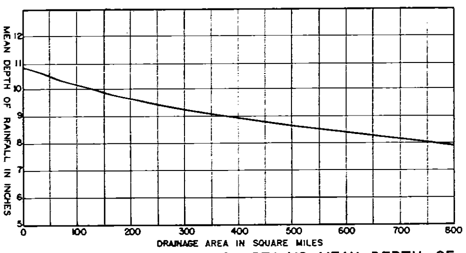


FIG. (2) GRAPH OF DRAINAGE AREA VS. MEAN DEPTH OF RAINFALL FOR THE STORM OF MARCH 17-19, 1936

SUMMER		WINTER	
IDENTIFICATION	VOLUME OF 1 HR. RAINFALL IN INCHES	IDENTIFICATION	VOLUME OF 1 HR. RAINFALL IN INCHES
A	3.90	F	0.80
B	3.80	G	0.75
C	3.65	H	0.70
D	3.50	J	0.60
E	3.40	K	0.50
ASSUMED TO APPLY FOR A DRAINAGE AREA OF 50 SQUARE MILES			

FIG. (6) MAXIMUM ONE HOUR DEPTH OF RAINFALL FROM AIR-MASS STUDY BY U.S. WEATHER BUREAU

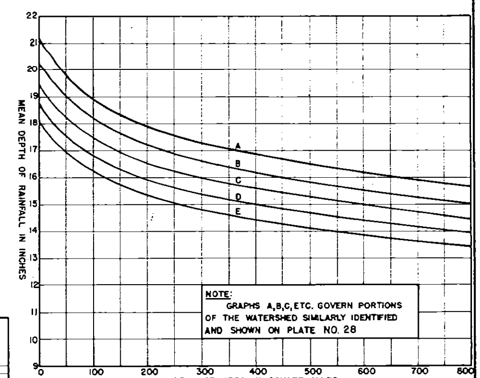


FIG. (10) DRAINAGE AREA VS. MEAN DEPTH OF RAINFALL FOR THE ADOPTED MAXIMUM SUMMER AND FALL SPILLWAY DESIGN STORM

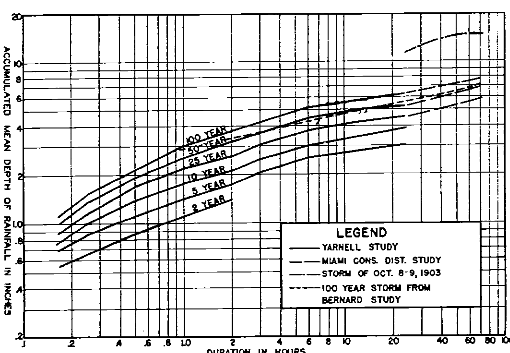


FIG. (3) DURATION VS. DEPTH OF RAINFALL FROM STUDIES OF RECORD

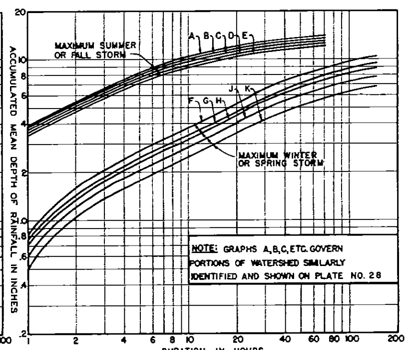


FIG. (7) DURATION VS. DEPTH OF RAINFALL FROM U.S. WEATHER BUREAU STUDY

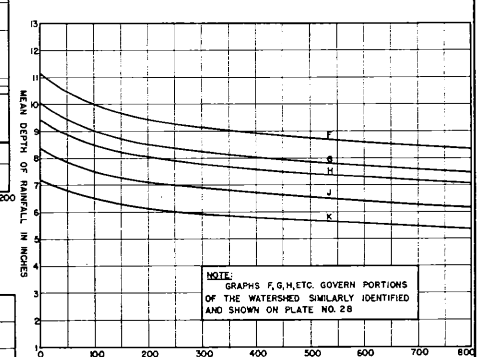


FIG. (11) DRAINAGE AREA VS. MEAN DEPTH OF RAINFALL FOR THE ADOPTED MAXIMUM WINTER AND SPRING SPILLWAY DESIGN STORM

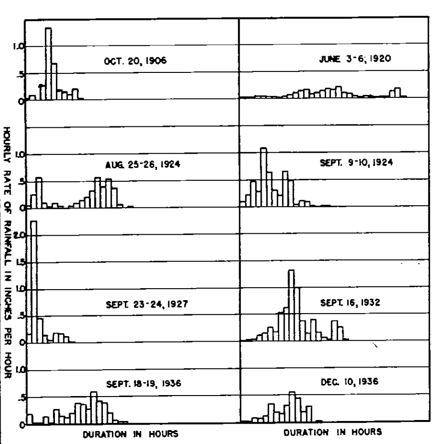


FIG. (4) RAINGRAPHS OF INTENSE STORMS AT PROVIDENCE, R. I.

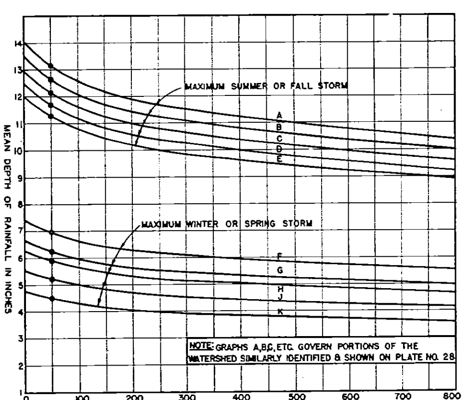


FIG. (8) DRAINAGE AREA VS. MEAN DEPTH OF RAINFALL FOR THE MAXIMUM 36 HOUR STORMS FROM THE U. S. WEATHER BUREAU STUDY

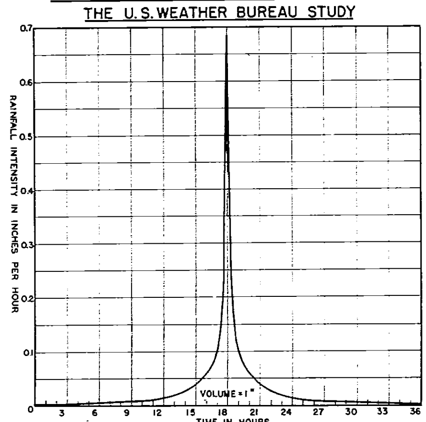


FIG. (9) INTENSITY VS. TIME FOR 1.0 INCH OF RAINFALL IN 36 HOURS

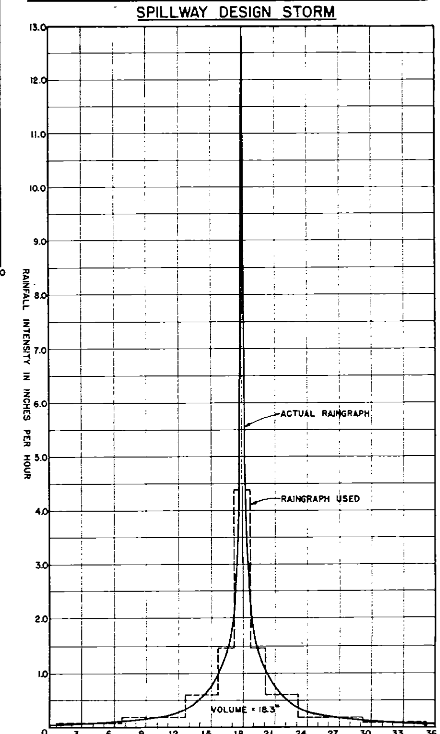


FIG. (12) GRAPH OF INTENSITY VS. TIME FOR TYPICAL SPILLWAY DESIGN STORM

CONNECTICUT RIVER FLOOD CONTROL
MAXIMUM STORM DATA OF RECORD
AND
ADOPTED SPILLWAY DESIGN STORMS

IN 4 SHEETS | SCALE AS SHOWN | SHEET NO. 2

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: H. T. McCarthy, ENGINEER | APPROVAL RECOMMENDED: Clarence E. [illegible], PRINCIPAL ENGINEER | APPROVED: [illegible], LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY R. M.
TRACED BY H. W. J.
CHECKED BY H.A.d.

TO ACCOMPANY REPORT DATED MARCH 20, 1937

FILE NO. CT-3/1070

CORPS OF ENGINEERS, U. S. ARMY

WAR DEPARTMENT

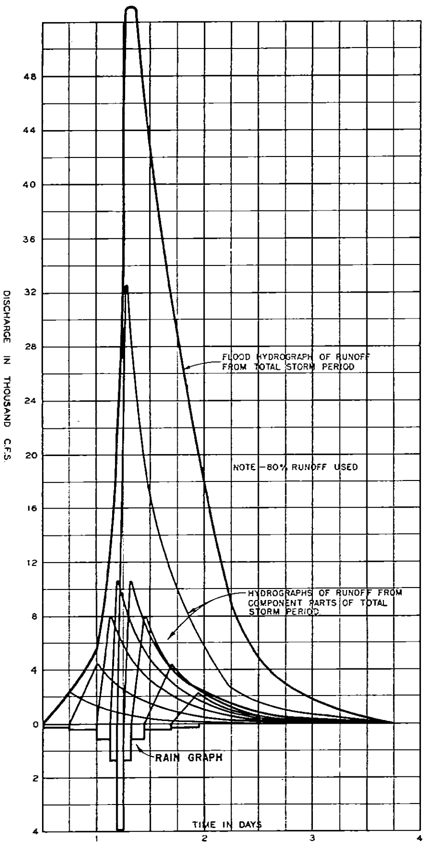


FIG (1) TYPICAL FLOOD HYDROGRAPHS
FROM MAXIMUM SUMMER OR FALL STORM

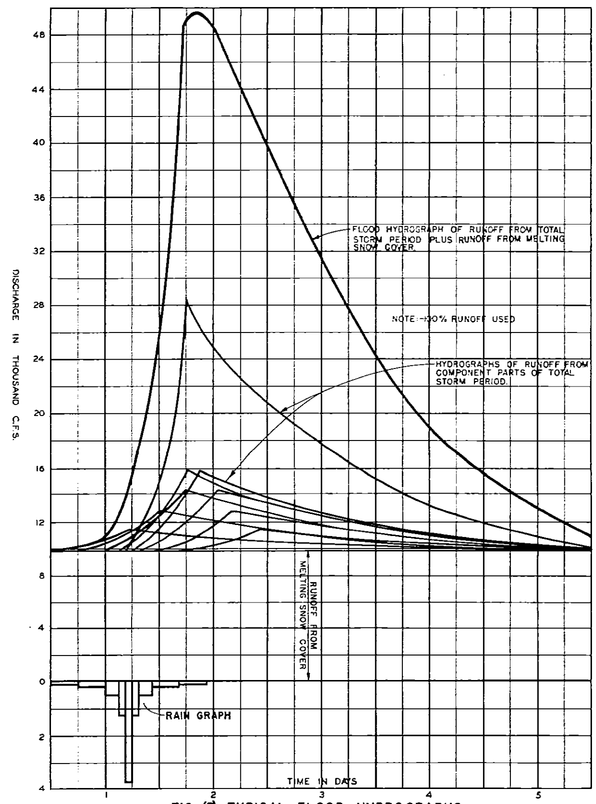


FIG (2) TYPICAL FLOOD HYDROGRAPHS
FROM MAXIMUM WINTER OR SPRING STORM ACCOMPANIED BY RUNOFF FROM MELTING SNOW COVER

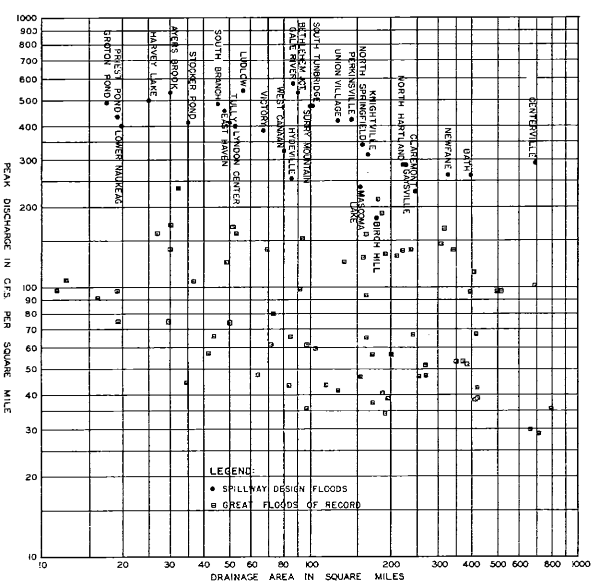


FIG. (3) RELATION OF DRAINAGE AREA TO PEAK DISCHARGE
FOR SPILLWAY DESIGN FLOODS AND MAXIMUM OBSERVED DISCHARGES OF RECORD IN THE CONNECTICUT RIVER BASIN

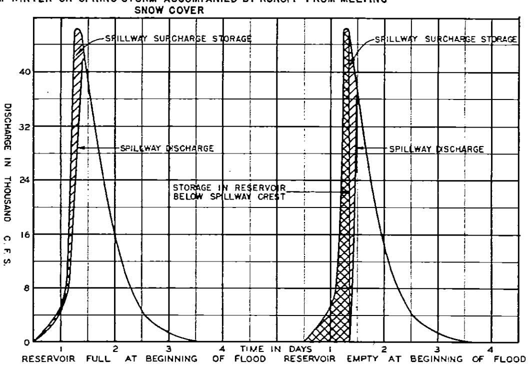


FIG (4) TYPICAL SPILLWAY DESIGN FLOOD HYDROGRAPHS
SHOWING THE EFFECT OF SMALL SPILLWAY SURCHARGE STORAGE UPON THE MAXIMUM SPILLWAY DISCHARGE

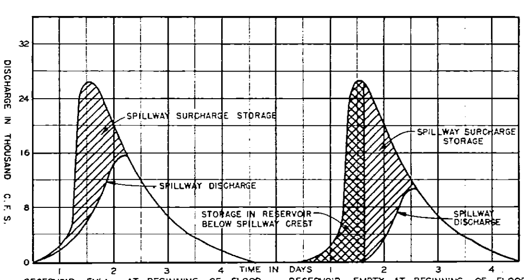


FIG (5) TYPICAL SPILLWAY DESIGN FLOOD HYDROGRAPHS
SHOWING THE EFFECT OF LARGE SPILLWAY SURCHARGE STORAGE UPON THE MAXIMUM SPILLWAY DISCHARGE

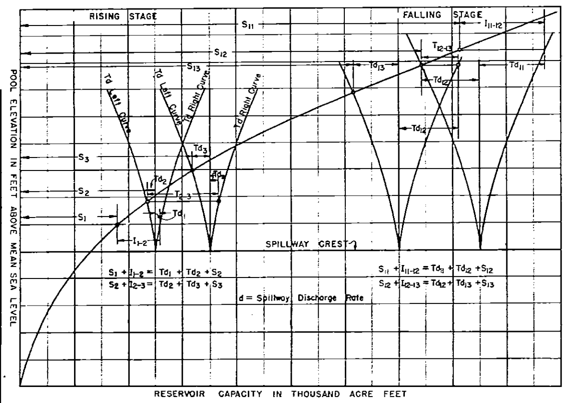


FIG. (6) TYPICAL DIAGRAMS FOR METHOD OF DETERMINING SPILLWAY SURCHARGES

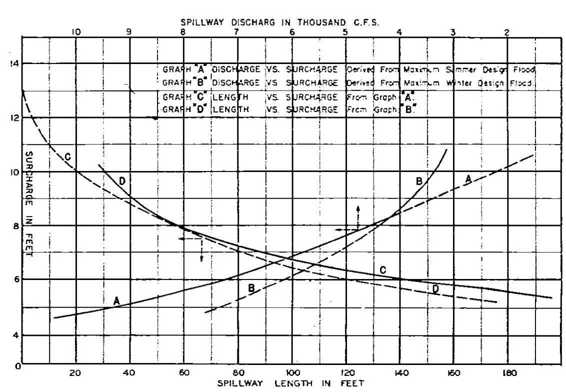


FIG (7) TYPICAL SPILLWAY SURCHARGES VERSUS DISCHARGE AND LENGTH CURVES

CONNECTICUT RIVER FLOOD CONTROL
SPILLWAY DESIGN FLOODS
MAXIMUM DISCHARGES,—TYPICAL HYDROGRAPHS AND FLOOD ROUTING

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAR., 1937

IN 4 SHEETS | SCALE AS SHOWN | SHEET NO 3

CT-3-1071

CORPS OF ENGINEERS, U.S. ARMY

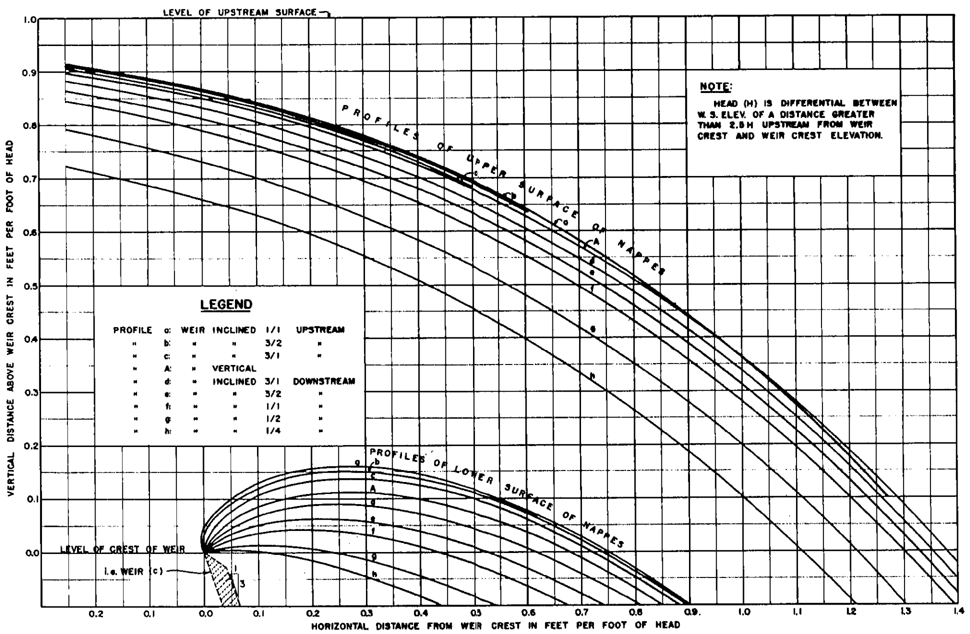


FIG. (1) EXPERIMENTAL SHARP – CRESTED WEIR JET PROFILES – H. BAZIN

FROM ANNALES DES PONTS ET CHAUSSÉES – OCTOBER 1888

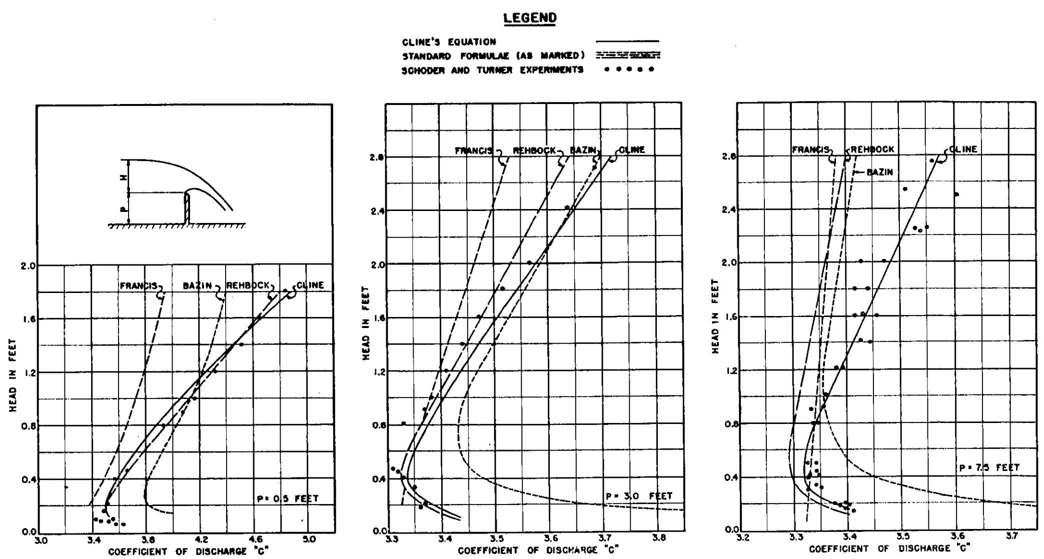


FIG. (3) COEFFICIENT OF DISCHARGE "C" FROM SHARP – CRESTED WEIR FORMULAE

AS GIVEN IN TRANS. A.S.C.E. VOL. 100, 1935, PAGE 407

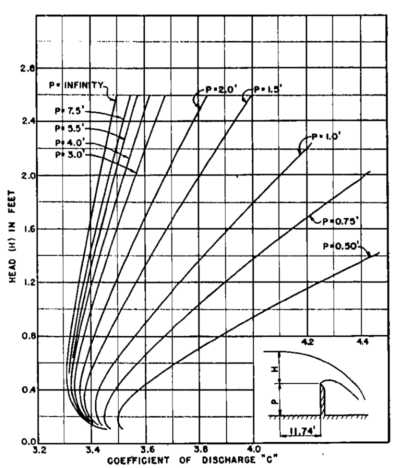


FIG. (2) RELATIONSHIP BETWEEN HEAD (H) AND "C" FOR SHARP – CRESTED WEIRS FROM CLINE'S EQUATION

FROM TRANS. A.S.C.E. VOL. 100, 1935, PAGE 408

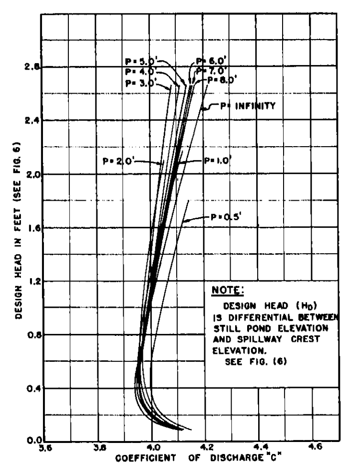


FIG. (4) RELATIONSHIP BETWEEN DESIGN HEAD AND "C" FOR OGEE DAM SECTION

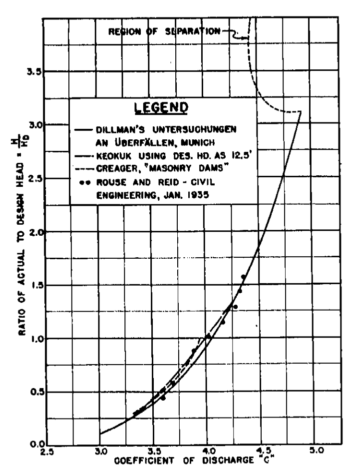


FIG. (5) RELATIONSHIP BETWEEN HEAD AND "C" FOR THEORETICAL OGEE DAM SECTION WITH VERTICAL UPSTREAM FACE

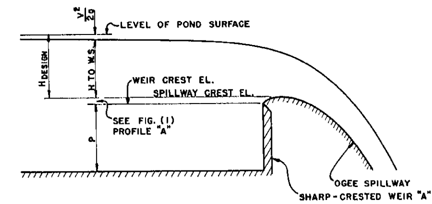
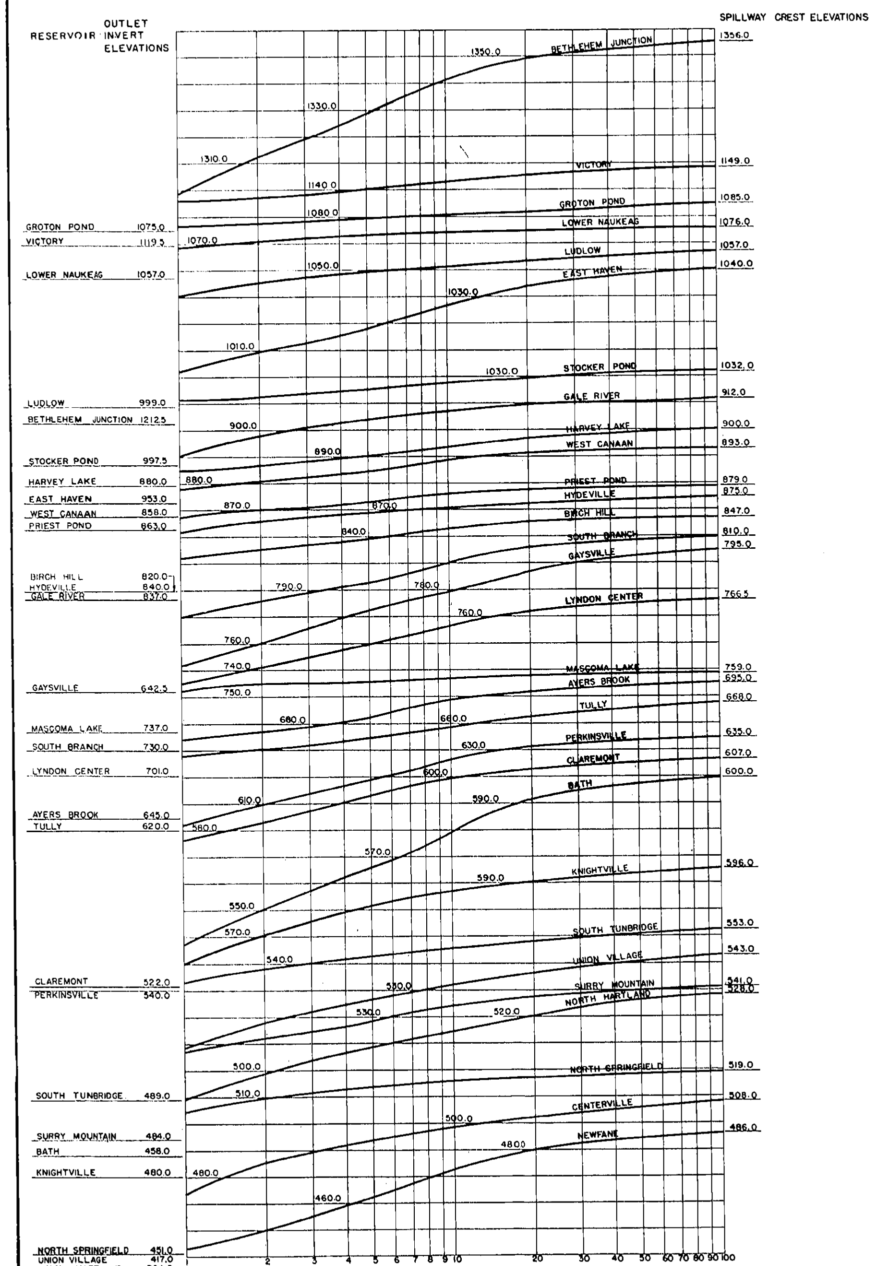


FIG. (6) SKETCH OF WATER SURFACE PROFILE OVER THEORETICAL OGEE SPILLWAY SECTION

CONNECTICUT RIVER FLOOD CONTROL

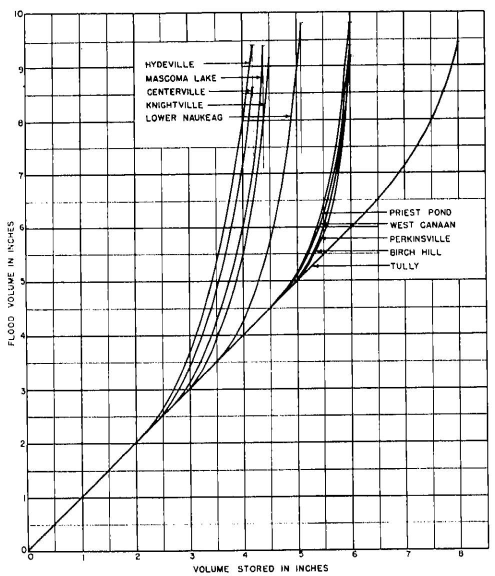
DISCHARGE COEFFICIENTS
FOR
OGEE SPILLWAYS

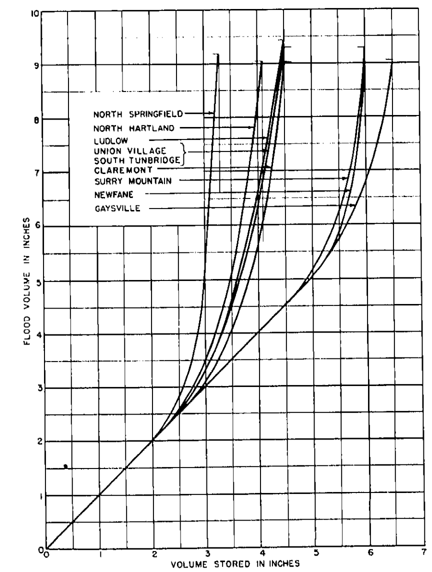
IN 4 SHEETS	SCALE AS SHOWN	SHEET NO. 4
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937		
SUBMITTED: G. T. McCarthy ENGINEER	APPROVAL RECOMMENDED: Clarence E. Kessel PRINCIPAL ENGINEER	APPROVED: Young LT. COL. CORPS OF ENGINEERS DISTRICT ENGINEER
DRAWN BY R. H. W. TRACED BY H. W. J. CHECKED BY	TO ACCOMPANY: REPORT DATED: MARCH 20, 1937	FILE NO. CT-3-1072



POOL ELEVATION VS FREQUENCY

NOTE: FIGURES SHOWN ON CURVES ARE ELEVATIONS IN FEET ABOVE MEAN SEA LEVEL.

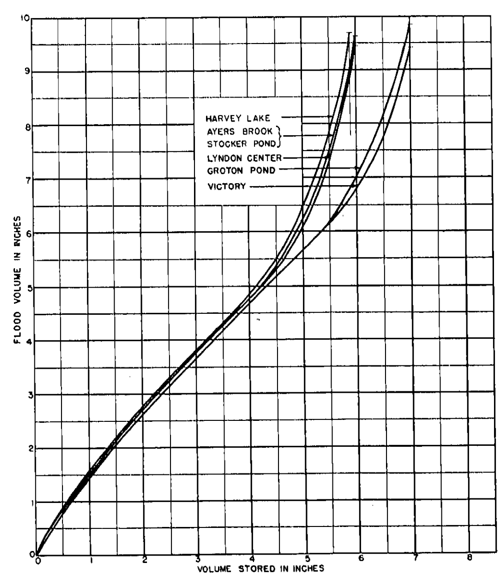


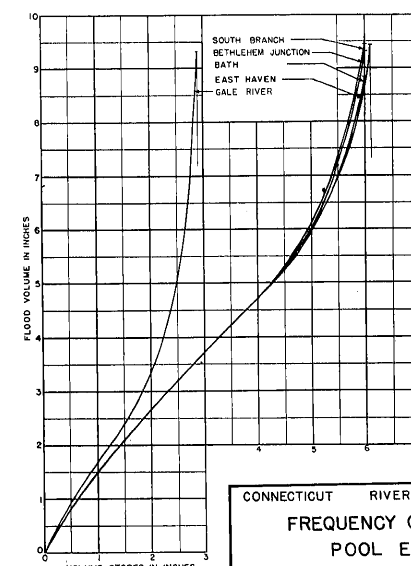


DETENTION BASINS

FLOOD VOLUME VS VOLUME STORED

NOTE: VERTICAL LINES INDICATE RESERVOIR CAPACITIES





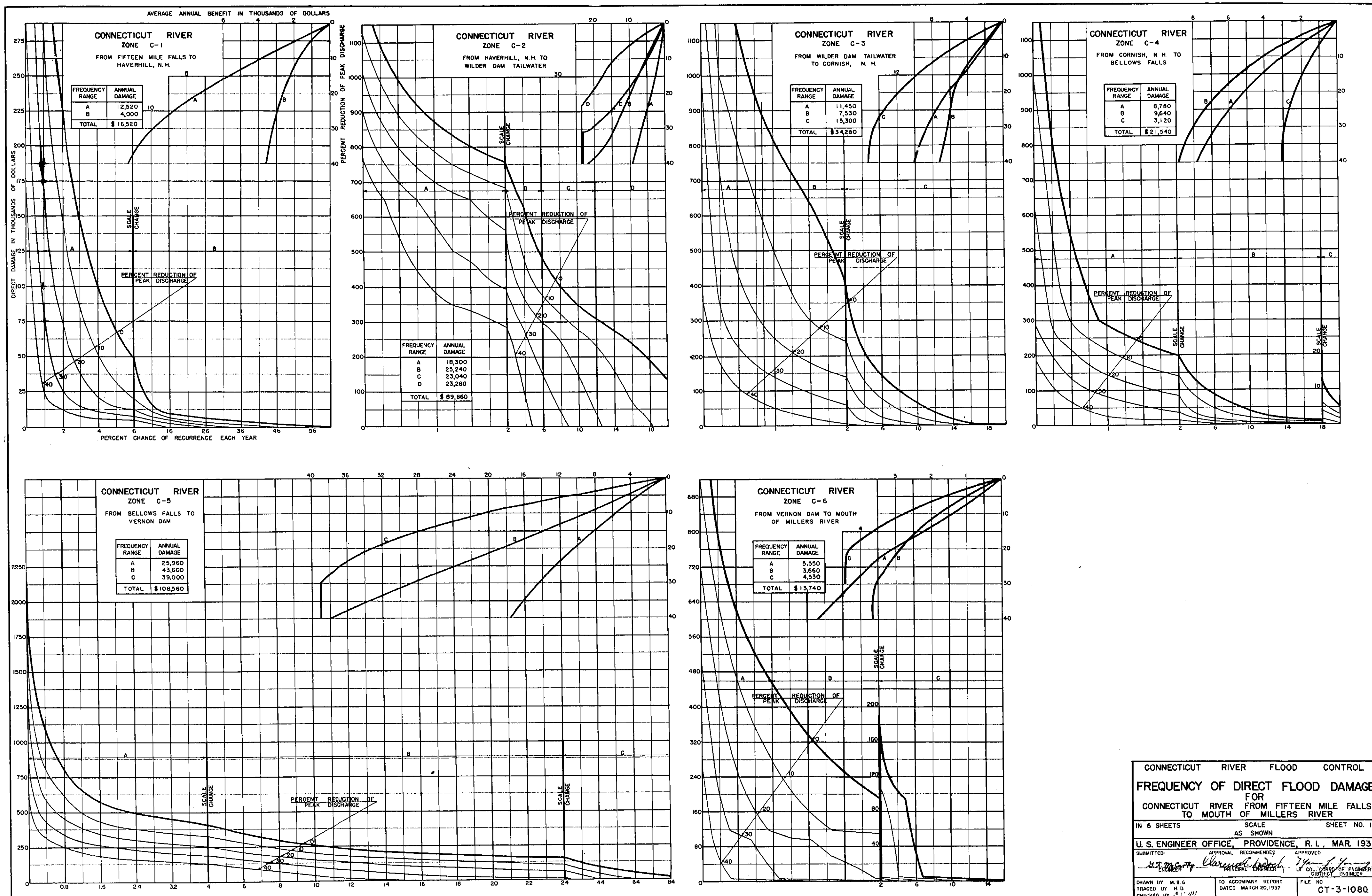
RETARDING BASINS

FLOOD VOLUME VS VOLUME STORED

CONNECTICUT RIVER FLOOD CONTROL

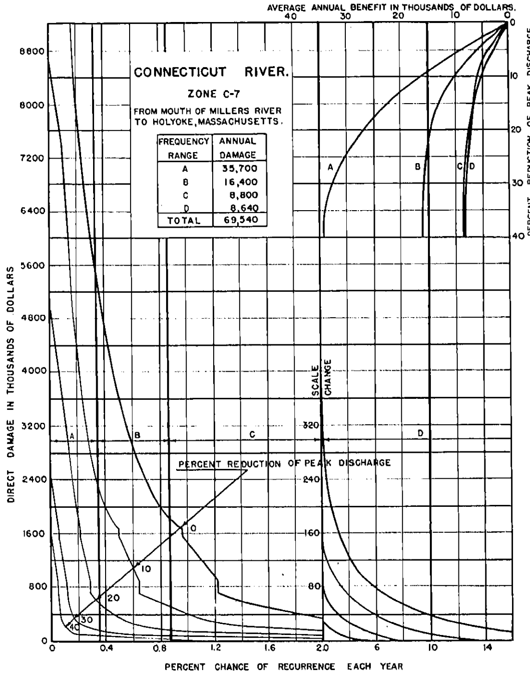
FREQUENCY OF RESERVOIR POOL ELEVATIONS

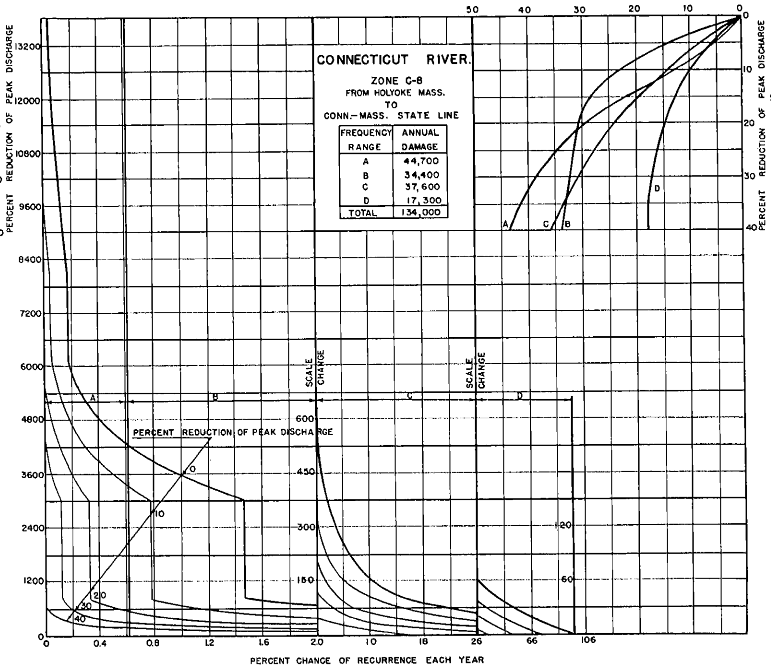
IN 1 SHEET	SCALE AS SHOWN	SHEET NO. 1
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.,		MAR. 1937
SUBMITTED: E. T. McCarthy, ENGINEER	APPROVAL RECOMMENDED: Clarence E. Bock, PRINCIPAL ENGINEER	APPROVED: Wm. L. Young, LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER
DRAWN BY R.N.W. TRACED BY H.D.G. H.E.W. CHECKED BY TMcF	TO ACCOMPANY REPORT DATED MARCH 20, 1937	FILE NO. CT-3-1074

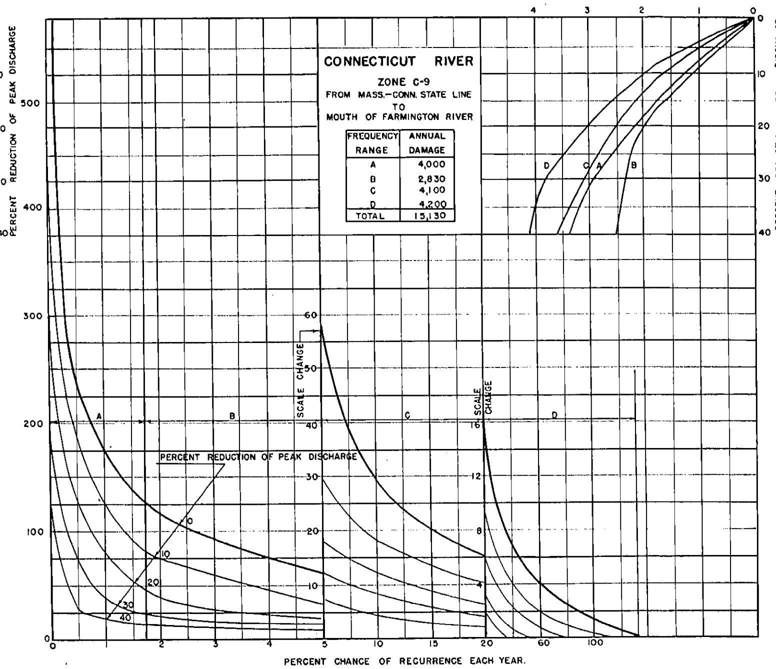

AVERAGE ANNUAL BENEFIT IN THOUSANDS OF DOLLARS
DIRECT DAMAGE IN THOUSANDS OF DOLLARS
PERCENT REDUCTION OF PEAK DISCHARGE
PERCENT CHANCE OF RECURRENCE EACH YEAR
CONNECTICUT RIVER
ZONE C-1
FROM FIFTEEN MILE FALLS TO HAVERHILL, N. H.
FREQUENCY RANGE | ANNUAL DAMAGE
A 12,520
B 4,000
TOTAL $ 16,520
CONNECTICUT RIVER
ZONE C-2
FROM HAVERHILL, N. H. TO WILDER DAM TAILWATER
FREQUENCY RANGE | ANNUAL DAMAGE
A 18,300
B 23,240
C 23,040
D 23,280
TOTAL $ 89,860
CONNECTICUT RIVER
ZONE C-3
FROM WILDER DAM TAILWATER TO CORNISH, N. H.
FREQUENCY RANGE | ANNUAL DAMAGE
A 11,450
B 7,530
C 15,300
TOTAL $34,280
CONNECTICUT RIVER
ZONE C-4
FROM CORNISH, N. H. TO BELLOWS FALLS
FREQUENCY RANGE | ANNUAL DAMAGE
A 8,780
B 9,640
C 3,120
TOTAL $21,540
CONNECTICUT RIVER
ZONE C-5
FROM BELLOWS FALLS TO VERNON DAM
FREQUENCY RANGE | ANNUAL DAMAGE
A 25,960
B 43,600
C 39,000
TOTAL $108,560
CONNECTICUT RIVER
ZONE C-6
FROM VERNON DAM TO MOUTH OF MILLERS RIVER
FREQUENCY RANGE | ANNUAL DAMAGE
A 5,550
B 3,660
C 4,530
TOTAL $ 13,740
SCALE CHANGE
CONNECTICUT RIVER FLOOD CONTROL
FREQUENCY OF DIRECT FLOOD DAMAGE
FOR
CONNECTICUT RIVER FROM FIFTEEN MILE FALLS TO MOUTH OF MILLERS RIVER
IN 6 SHEETS
SCALE AS SHOWN
SHEET NO. 1
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MAR. 1937
SUBMITTED:
APPROVAL RECOMMENDED:
APPROVED:
ENGINEER
PRINCIPAL ENGINEER
COL. CORPS OF ENGINEERS DISTRICT ENGINEER
DRAWN BY M.S.G.
TRACED BY H.D.
TO ACCOMPANY REPORT DATED MARCH 20, 1937
FILE NO. CT-3-1080

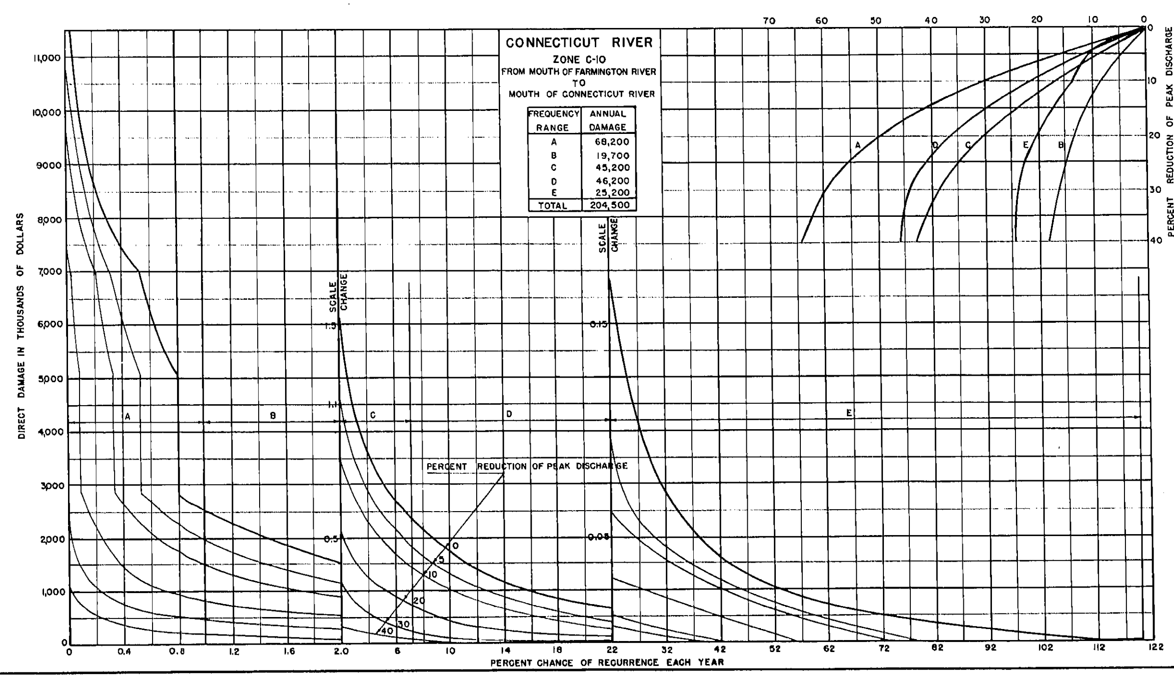
WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY.









CONNECTICUT RIVER FLOOD CONTROL

FREQUENCY OF DIRECT FLOOD DAMAGE.
FOR
CONNECTICUT RIVER BELOW MOUTH OF MILLERS RIVER

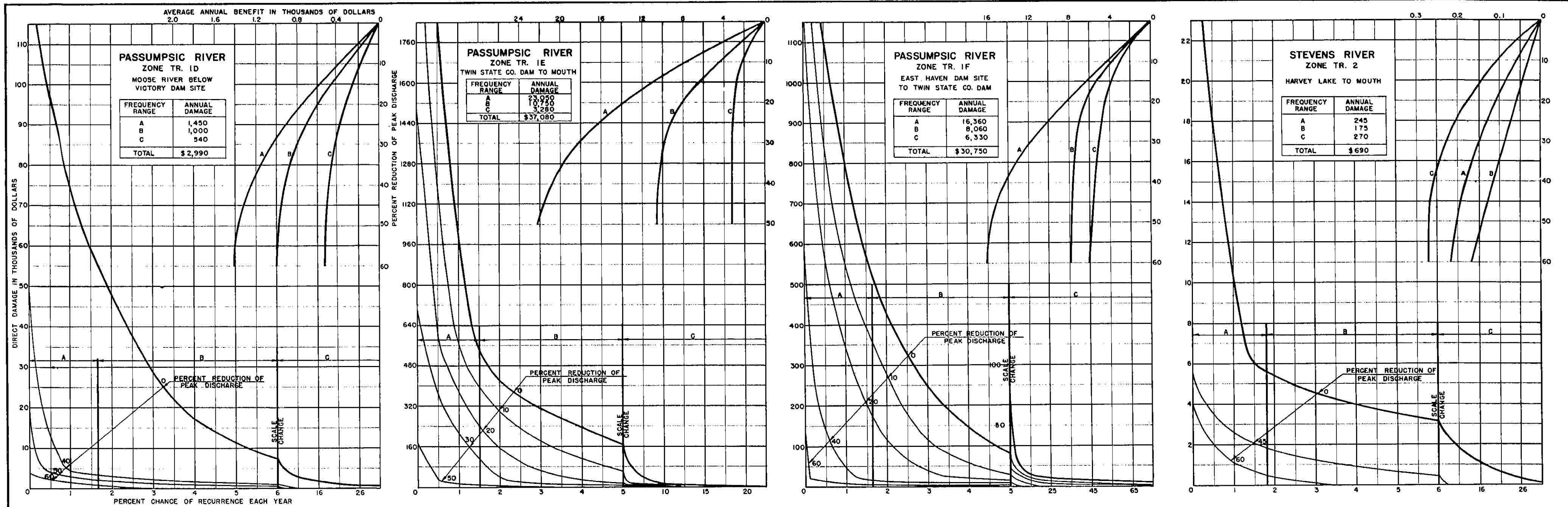
IN 6 SHEETS | SCALE AS SHOWN | SHEET NO. 2

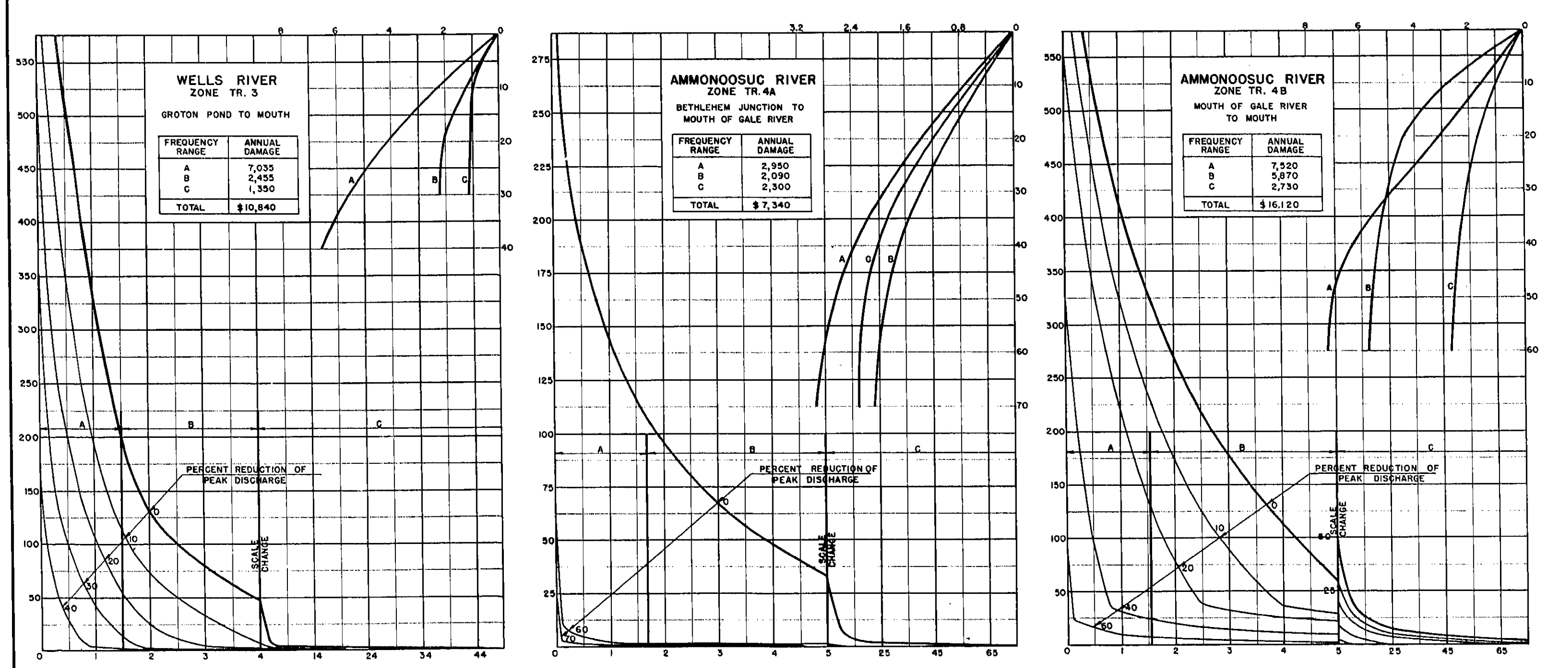
U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937.

SUBMITTED: G.T. McCarthy, ENGINEER | APPROVAL RECOMMENDED: Clarence Blood, PRINCIPAL ENGINEER | APPROVED: LT. COL. CORPS OF ENGINEERS

DRAWN BY: M.S.G. TRACED BY: V.A.T. CHECKED BY: G.T.M. | TO ACCOMPANY REPORT DATED MARCH 20, 1937 | FILE NO. CT-3-1079

SEC. I - PLATE NO. 44





CONNECTICUT RIVER FLOOD CONTROL

FREQUENCY OF DIRECT FLOOD DAMAGE FOR PASSUMPSIC, STEVENS, WELLS, & AMMONOOSUC RIVERS

IN 6 SHEETS | SCALE AS SHOWN | SHEET NO. 3

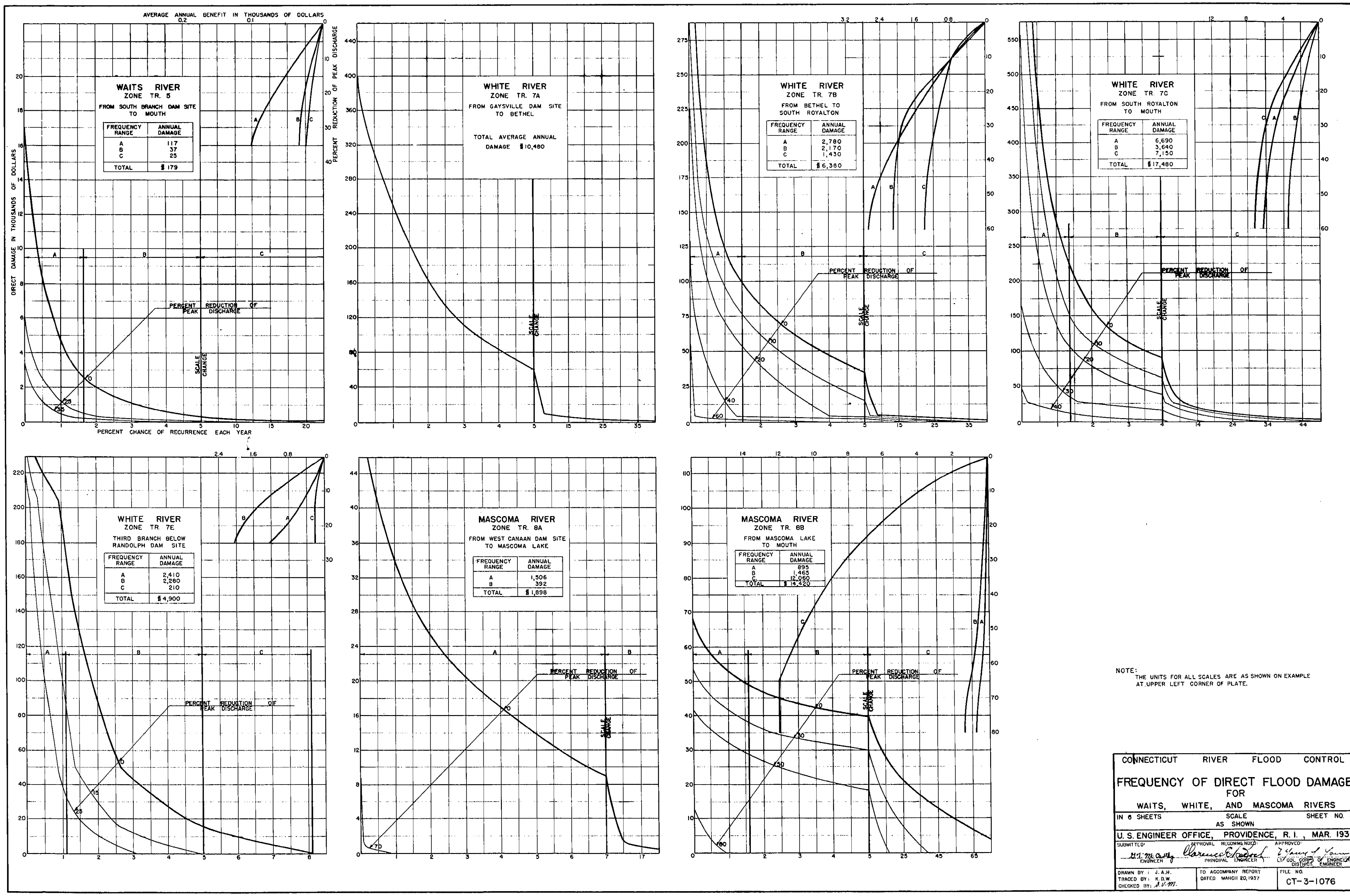
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: ENGINEER — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: K.E.
TRACED BY: H.W.J.
CHECKED BY:

TO ACCOMPANY REPORT DATED MARCH 20, 1937

FILE NO. CT-3-1075



NOTE:
THE UNITS FOR ALL SCALES ARE AS SHOWN ON EXAMPLE AT UPPER LEFT CORNER OF PLATE.

CONNECTICUT RIVER FLOOD CONTROL

FREQUENCY OF DIRECT FLOOD DAMAGE FOR WAITS, WHITE, AND MASCOMA RIVERS

IN 6 SHEETS | SCALE AS SHOWN | SHEET NO. 4

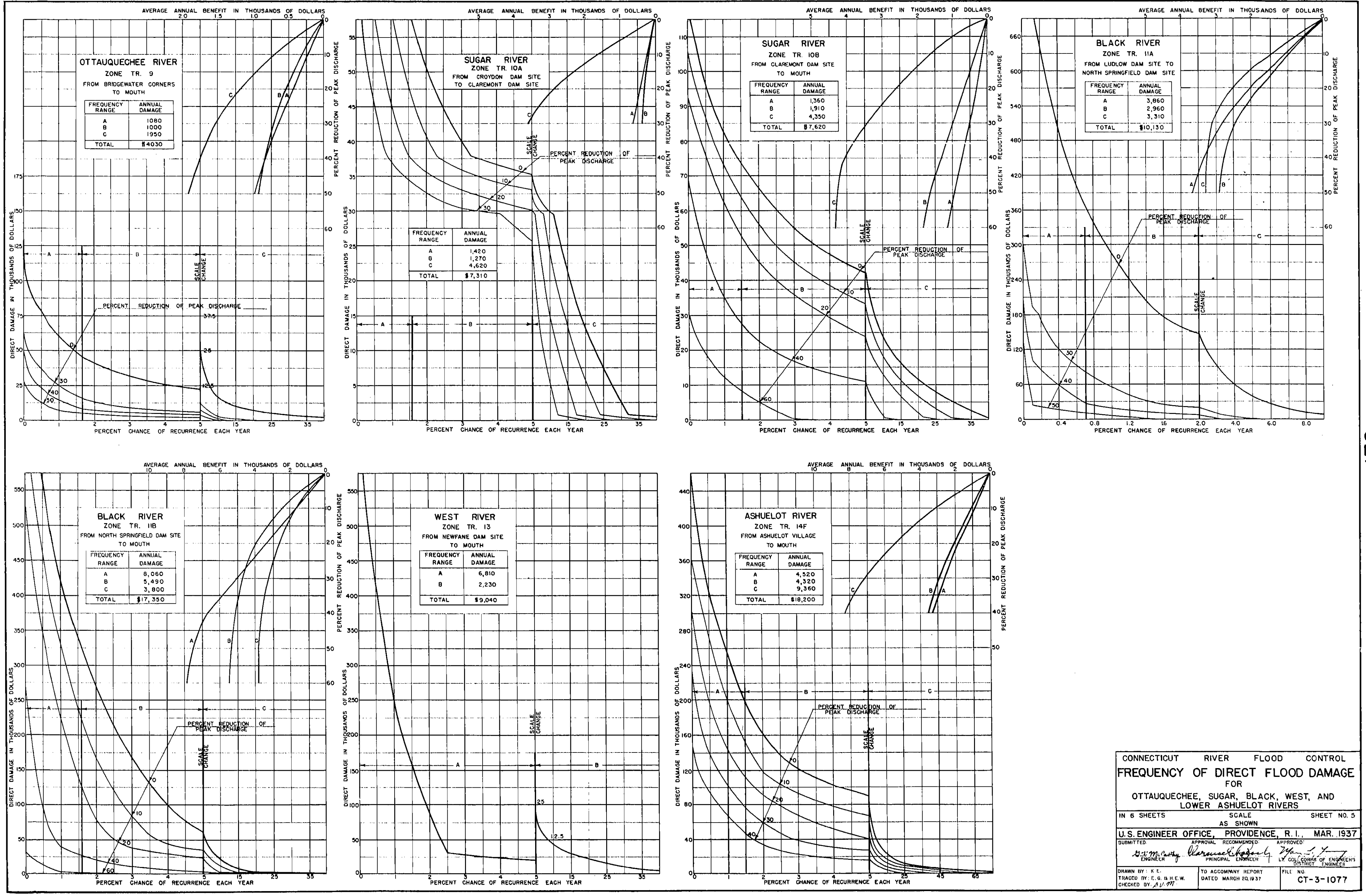
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MAR. 1937

SUBMITTED: G.T. McCarthy, ENGINEER | APPROVAL RECOMMENDED: Clarence E. Kaskell, PRINCIPAL ENGINEER | APPROVED: J. Young, LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: J. A.M. TRACED BY: K. D.W. CHECKED BY: J.V.M. | TO ACCOMPANY REPORT DATED MARCH 20, 1937 | FILE NO. CT-3-1076

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



CONNECTICUT RIVER FLOOD CONTROL

FREQUENCY OF DIRECT FLOOD DAMAGE

FOR

OTTAUQUECHEE, SUGAR, BLACK, WEST, AND LOWER ASHUELOT RIVERS

IN 6 SHEETS | SCALE AS SHOWN | SHEET NO. 5

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

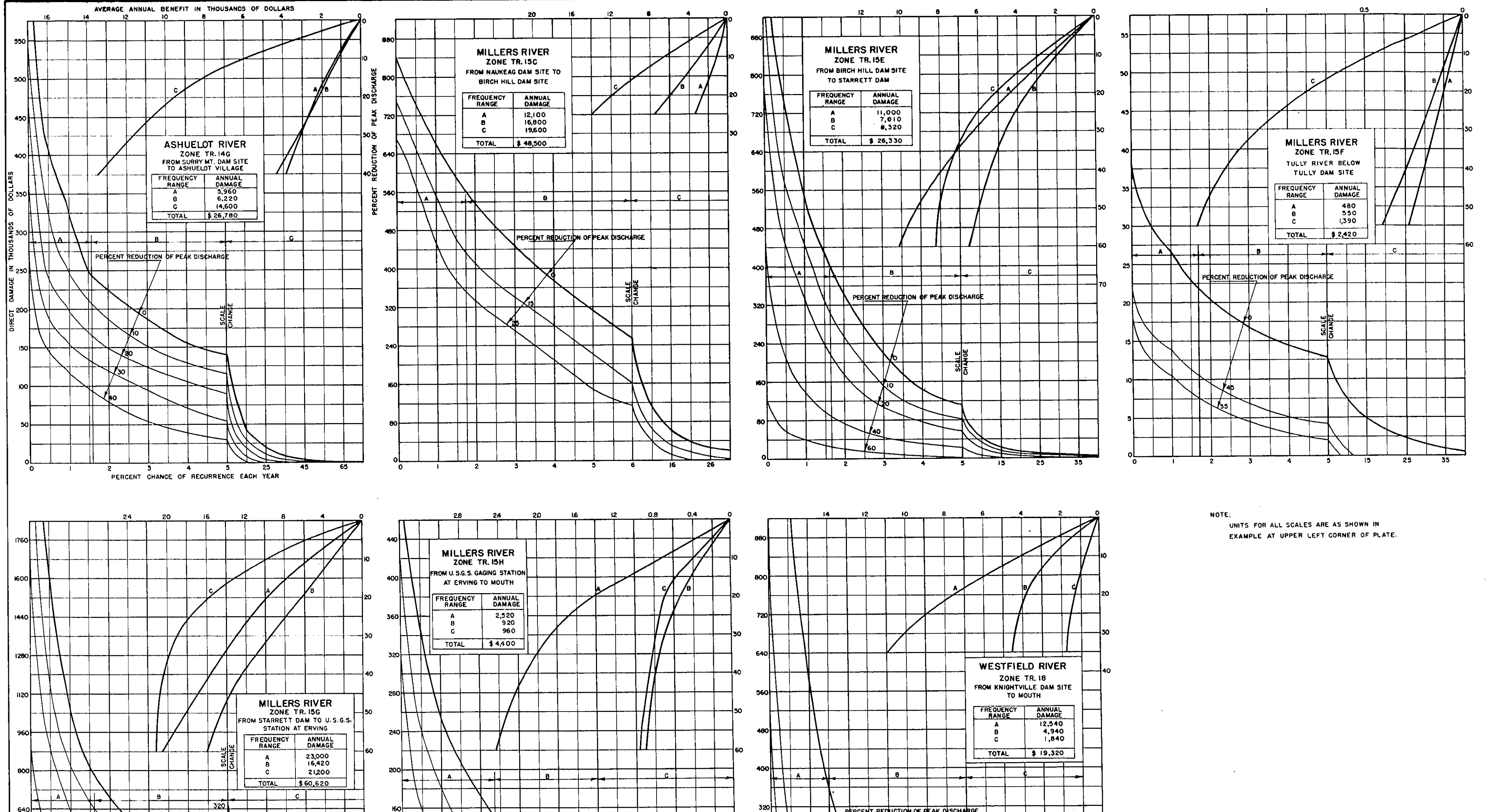
SUBMITTED: G.W. McCarthy, ENGINEER | APPROVAL RECOMMENDED: Clarence ... , PRINCIPAL ENGINEER | APPROVED: ..., LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: I. K. L.
TRACED BY: E. G. & H.E.W.
CHECKED BY: N.V.M.

TO ACCOMPANY REPORT DATED MARCH 20, 1937

FILE NO. CT-3-1077

SEC. I - PLATE NO. 47



NOTE:
UNITS FOR ALL SCALES ARE AS SHOWN IN EXAMPLE AT UPPER LEFT CORNER OF PLATE.

CONNECTICUT RIVER FLOOD CONTROL

FREQUENCY OF DIRECT FLOOD DAMAGE
FOR
UPPER ASHUELOT, MILLERS, AND WESTFIELD RIVERS

IN 6 SHEETS — SCALE AS SHOWN — SHEET NO. 6

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED: G. J. McCarthy, ENGINEER — APPROVAL RECOMMENDED: Clarence L. Jacobs, PRINCIPAL ENGINEER — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY: J.A.H. TRACED BY: E.G. CHECKED BY: — TO ACCOMPANY REPORT DATED MARCH 20, 1937 — FILE NO. CT-3-1078

SECTION II

TABLE REFERENCE

TABLE 16

DIRECT LOSSES - CONNECTICUT RIVER WATERSHED

SUMMARY OF 1927 LOSSES BY STATES

1927 Direct Flood Losses in Thousands of Dollars							
STATE	Urban	Rural	Industrial	Highway	Railroad	Total	Per Cent
Vermont	1,730	169	1,181	4,960	2,391	10,931	70.7
New Hampshire	115	76	110	1,336	130	1,767	11.4
Massachusetts	505	275	507	295	575	2,157	13.9
Connecticut	275	145	126	0	75	621	4.0
TOTALS	2,675	665	1,924	6,591	3,671	15,526	100.0
PER CENT	17.2	4.3	12.4	42.5	23.6	100.0	

TABLE 17

DIRECT LOSSES - CONNECTICUT RIVER WATERSHED

SUMMARY OF 1927 LOSSES BY RIVER BASINS

River Basin	State	Damages (in thousands of dollars)					
		Urban	Rural	Industrial	Railroad	Highway	Total
Connecticut *	Various	846	565	1,155	887	554	4,007
Israel	New Hampshire		2	10	5	45	62
Passumpsic	Vermont	563	50	185	475	1,311	2,584
Ammonoosuc	New Hampshire	35	6		110	829	980
Stevens	Vermont			1		22	23
Wells	Vermont	98	3	60	186	289	636
Waits	Vermont					56	56
Ompompanoosuc	Vermont		1	4		88	93
White **	Vermont	751		250	1,195	1,935	4,131
Ottauquechee	Vermont	16	10	21	20	465	532
Black	Vermont	251	20	120	130	195	716
Williams	Vermont					63	63
Saxtons	Vermont					59	59
West	Vermont		8	10	113	370	501
Westfield ***	Massachusetts	100		32	550	260	942
Farmington	Connecticut	15		26			41
Total		2,675	665	1,924	3,671	6,591	15,526
Percent of total		17.2	4.3	12.4	23.6	42.5	100.0

* Exclusive of tributaries listed in table. There were 6 lives lost.
** There were 9 lives lost in White Basin.
*** There were 6 lives lost in Westfield Basin.

TABLE 18

DIRECT FLOOD LOSSES - CONNECTICUT RIVER WATERSHED - 1936 FLOOD
STATE OF VERMONT

Summary of Direct Losses and Assessed Valuations of Towns Reporting Losses.

(Note: *identifies losses not subject to control by Studied Reservoir Plans. Number in column (1) refers to damage zones.)

Town	Assessment 1935 Grand List Value	Direct Flood Loss					
		Urban	Rural	Indus-trial	Highway	Rail-road	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
*Andover	$ 177,625	$ -	$ -	$ -	$ 6,725	$ -	$ 6,725
*Athens	104,910	-	-	-	3,000	-	3,000
*Averill	200,000	-	-	-	50	-	50
*Baltimore	38,605	-	-	-	100	-	100
Barnet, C-1		3,200	1,530	-	500	18,400	23,630
Barnet, TR-1E		2,100	2,760	7,000	0	5,800	17,660
Barnet, TR-2		0	0	0	3,000	-	3,000
Barnet, Totals	2,662,797	5,300	4,290	7,000	3,500	24,200	44,290
Bethel	1,020,656	0	0	0	7,950	0	7,950
* Bloomfield	315,749	-	-	-	850	-	850
Bradford, C-2		5,500	4,235	4,000	200	844	14,779
Bradford, TR-5		200	0	0	0	0	200
Bradford, Totals	1,102,979	5,700	4,235	4,000	200	844	14,979
Brattleboro, C-5		3,000	825	108,500	35,000	50,500	197,825
Brattleboro, TR-13		0	800	2,000	5,000	0	7,800
Brattleboro, Totals	8,213,900	3,000	1,625	110,500	40,000	50,500	205,625
Bridgewater, TR-9		120	0	2,200	0	0	2,320
*Bridgewater		500	-	500	-	-	1,000
Bridgewater, Totals	601,627	620	0	2,700	0	0	3,320
*Brighton	973,515	-	-	-	100	-	100
*Brookline	72,575	-	-	-	2,500	-	2,500
Burke	682,363	500	0	0	0	0	500
*Canaan	838,789	-	-	-	2,550	-	2,550
Cavendish	1,060,425	500	0	500	4,600	2,500	8,100
*Chester	1,394,014	-	-	-	3,100	-	3,100
Concord	830,295	-	0	-	0	2,000	2,000
*Corinth	476,715	-	-	-	1,600	-	1,600
*Dover	287,496	-	-	-	3,300	-	3,300

TABLE 18

Town (1)	1935 Grand List Value (2)	Urban (3)	Rural (4)	Indus-trial (5)	Highway (6)	Rail-road (7)	Total (8)
Dummerston, C-5	-	$ 600	$ 3,465	0	-	$63,000	$ 67,065
Dummerston, TR-13	-	0	1,835	7,500	15,000	0	24,335
Dummerston, Totals	$561,850	600	5,300	7,500	15,000	63,000	91,400
Fairlee	936,629	0	2,120	0	75,000	2,150	79,270
*Glastenbury	63,952	-	-	-	150	-	150
*Grafton	343,034	-	-	-	3,000	-	3,000
Groton	547,400	500	0	800	0	0	1,300
*Guildhall	271,957	-	-	-	100	-	100
*Guilford	464,547	-	-	-	7,800	-	7,800
*Halifax	225,427				20,000		20,000
Hartford, C-2		-	0	-	-	-	0
Hartford, C-3		9,800	0	24,600	0	2,300	36,700
Hartford, TR-9		0	0	6,000	0	0	6,000
Hartford, TR-7C		300	720	1,600	900	0	3,520
Hartford, Totals	4,250,773	10,100	720	32,200	900	2,300	46,220
Hartland, C-3		0	5,605	0	0	0	5,605
Hartland, TR-9		0	0	0	3,200	0	3,200
Hartland, Totals	886,546	0	5,605	0	3,200	0	8,805
*Jamaica	344,865	-	-	-	50,500	-	50,500
Kirby	218,128	-	-	-	-	-	0
*Landgrove	70,815	-	-	-	800	-	800
*Londonderry	468,785	-	-	-	9,500	-	9,500
Ludlow, TR-11A		2,000	0	9,000	4,700	2,500	18,200
*Ludlow		-	-	-	4,700	-	4,700
Ludlow, Totals	1,539,785	2,000	0	9,000	9,400	2,500	22,900
Lyndon	2,018,770	1,500	3,100	6,000	1,000	200	11,800
*Marlboro	198,210	-	-	-	13,000	-	13,000
Newbury, C-1		2,400	9,400	700	0	2,930	15,430
Newbury, TR-3		0	0	1,400	0	0	1,400
Newbury, Totals	1,454,300	2,400	9,400	2,100	0	2,930	16,830
Newfane	451,525	0	0	0	5,000	0	5,000
Norwich, C-2		4,200	3,120	4,400	0	21,400	33,120
*Norwich					650		650
Norwich Totals	1,060,830	4,200	3,120	4,400	650	21,400	33,770
*Peru	148,783	-	-	-	1,200	-	1,200
*Plymouth	322,826	-	-	-	2,000	-	2,000
Pomfret	526,775	0	0	0	0	0	0
Putney	607,656	400	15,725	500	5,000	45,800	67,425
Randolph	2,401,640	0	0	0	0	0	0
*Reading	393,000		-	-	2,300	-	2,300
*Readsboro	928,624				3,000		3,000
Rockingham, C-4		-	7,185	-	115,000	20,700	142,885
Rockingham, C-5		5,600	0	41,500	0	0	47,100
*Rockingham, TR-12		0	0	0	3,290	-	3,290
Rockingham, Totals	10,725,688	5,600	7,185	41,500	118,290	20,700	193,275

TABLE 18

Town (1)	1935 Grand List Value (2)	Urban (3)	Rural (4)	Industrial (5)	Highway (6)	Railroad (7)	Total (8)
Royalton, TR-7B		100	240	0	4,000	0	4,340
Royalton, TR-7C		0	190	0	0	0	190
Royalton, Totals	$1,048,281	100	430	0	4,000	0	4,530
Ryegate, C-1		1,000	1,965	10,000	0	3,500	16,465
Ryegate, TR-3		1,100	0	1,000	0	0	2,100
Ryegate, Totals	1,034,448	2,100	1,965	11,000	0	3,500	18,565
*Searsburg	594,914	-	-	-	1,000	-	1,000
Sharon	366,513	0	1,230	0	350	24,000	25,580
*Somerset	554,897	-	-	-	1,100	-	1,100
Springfield, C-4		-	13,565	-	37,350	0	50,915
Springfield, TR-11B		6,500	650	2,800	30,600	0	40,550
Springfield, Totals	9,438,052	6,500	14,215	2,800	67,950	0	91,465
St.Johnsbury, TR-1F		4,700	190	1,800	2,500	600	9,790
St.Johnsbury, TR-1D		-	-	5,000	2,500	-	7,500
St.Johnsbury, TR-1E		3,600	0	18,000	0	1,200	22,800
St.Johnsbury, Totals	7,522,676	8,300	190	24,800	5,000	1,800	40,090
Stockbridge	339,133	0	0	0	0	0	0
Thetford	708,672	3,500	2,140	0	25,000	13,600	44,240
*Townshend	443,093	-	-	-	12,000	-	12,000
Vernon, C-5		0	825	20,000	0	0	20,825
Vernon, C-6		-	3,250	1,000	2,500	9,400	16,150
Vernon, Totals	955,227	0	4,075	21,000	2,500	9,400	36,975
Victory	248,250	-	0	-	-	-	0
*Wardsboro	191,600	-	-	-	10,000	-	10,000
*Washington	329,367	-	-	-	2,000	-	2,000
*Waterford	1,202,565	-	-	-	700	-	700
Weathersfield, C-4		0	1,895	0	23,600	0	25,495
Weathersfield, TR-11A		0	250	0	0	0	250
Weathersfield, Totals	825,941	0	2,145	0	23,600	0	25,745
Westminster, C-5		0	76,755	0	50,000	51,500	178,255
*Westminster, TR-12		0	0	0	-	-	0
Westminster, Totals	855,291	0	76,755	0	50,000	51,500	178,255
*Weston	264,400	-	-	-	700	-	700
*Whitingham	3,985,841	-	-	-	10,500	-	10,500
*Wilmington	2,628,998	-	-	-	5,000	64,400	69,400
*Windham	198,125	-	-	-	6,000	-	6,000
Windsor	4,008,293	34,500	12,040	40,000	2,000	32,100	120,640
*Winhall	233,156	-	-	-	2,000	-	2,000
*Woodford	227,124	-	-	-	10,000	-	10,000
Woodstock	2,668,143	600	0	0	0	0	600
Total (Vermont Towns)	94,416,155	98,520	177,610	328,300	673,315	441,324	1,719,069
*Estimate of Losses not included above		2,480	5,390	8,700	17,685	11,676	45,931
Grand Total, Vermont		101,000	183,000	337,000	691,000	453,000	1,765,000
20-Reservoir Plan:							
Below Reservoirs	68,562,277	94,800	177,360	310,100	448,950	371,924	1,403,134
Above Reservoirs		6,200	5,640	26,900	242,050	81,076	361,866

"1935 Grand List Value" from "1936 Vermont Year Book."

TABLE 19

DIRECT FLOOD LOSSES - CONNECTICUT RIVER WATERSHED - 1936 FLOOD

STATE OF NEW HAMPSHIRE

SUMMARY OF DIRECT LOSSES AND ASSESSED VALUATIONS OF TOWNS REPORTING LOSSES

(NOTE: * IDENTIFIES LOSSES NOT SUBJECT TO CONTROL BY STUDIED RESERVOIR PLANS. NUMBER IN COLUMN (1) REFERS TO DAMAGE ZONES.)

TOWN	1935 ASSESSED VALUE	DIRECT FLOOD LOSS					
		URBAN	RURAL	INDUSTRIAL	HIGHWAY	RAILROAD	TOTAL
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
*ACWORTH	$ 355,800	$	$ 700	$	$ 8,100	$	$ 8,800
*ALSTEAD	761,020		100		17,600		17,700
BATH, TR-4B		0	5,470	7,960	1,000	2,900	17,330
BATH, C-1		0	200	0	18,000	0	18,200
BATH, TOTAL	923,720	0	5,670	7,960	19,000	2,900	35,530
*BENTON	168,480				32,800		32,800
BETHLEHEM	3,164,139	0	1,030	0	2,300	0	3,330
CANAAN, TR-8A						11,800	11,800
*CANAAN				1,700	29,600		31,300
CANAAN, TOTAL	1,115,520			1,700	29,600	11,800	43,100
*CARROLL	1,499,475				5,000	1,000	6,000
CHARLESTOWN	1,862,505	0	35,480	0	6,900	22,900	65,280
CHESTERFIELD	1,301,689	0	6,565	100	157,500	0	164,165
CLAREMONT, TR-10A		0	310	2,000	0	3,400	5,710
CLAREMONT, TR-10B		1,300	0	20,465	29,600	0	51,365
CLAREMONT, C-4		0	6,800	0	0	0	6,800
CLAREMONT, TOTAL	13,991,480	1,300	7,110	22,465	29,600	3,400	63,875
*CLARKSVILLE	519,210				2,000		2,000
*COLEBROOK	1,927,383		90	500	2,200	1,500	4,290
*COLUMBIA	539,560		125	500	800		1,425
CORNISH	939,692	0	3,500	0	17,100	0	20,600
CROYDON, TR-10A		0	0	0	0	0	0
*CROYDON					20,000		20,000
CROYDON, TOTAL	417,234	0	0	0	20,000	0	20,000
*DALTON	439,514				3,800		3,800
*DORCHESTER	243,067				2,100		2,100
*EASTON	143,937				4,000		4,000
ENFIELD 8A	1,317,843	500		2,200	3,700		6,400
*FITZWILLIAM	837,920				5,000		5,000
*FRANCONIA	1,034,655				15,000		15,000
*GILSUM	294,910			4,000	16,600		20,600
*GOSHEN	278,753				600		600
*GRANTHAM	219,775				4,800		4,800
HANOVER	5,503,389	0	0	0	1,550	0	1,550
*HARRISVILLE	916,208				10,700		10,700
HAVERHILL, C-1		6,700	14,840	1,500	5,700	1,700	30,440
HAVERHILL, TR-4B				1,000			1,000
HAVERHILL, TOTAL	$3,674,913	$6,700	$14,840	$2,500	$ 5,700	$1,700	$31,440

Town	1935 Assessed Value	Direct Flood Loss: Urban	Rural	Industrial	Highway	Railroad	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Hinsdale, C-5	$	$ 0	$ 0	$ 61,000	$ 11,500	$ 70,000	$ 142,500
Hinsdale, C-6			2,850		15,000	0	17,850
Hinsdale, TR-14F		2,950	7,915	149,763	11,000	5,700	177,328
Hinsdale, Total	3,391,675	2,950	10,765	210,763	37,500	75,700	337,678
*Jefferson	934,409				23,100	1,100	24,200
*Keene, Above Zone		1,000	0	0	2,200	0	3,200
Keene, TR-14G		18,250	3,830	91,204	14,000	1,100	123,434
Keene, Total	17,360,504	19,250	3,830	91,204	16,200	1,100	131,634
*Lancaster	2,958,654	500	4,010	4,440	4,700		13,650
*Landoff	325,034				8,100		8,100
*Langdon	216,078				5,900		5,900
Lebanon, C-2		0	0	0	0	0	0
Lebanon, C-3		3,600	5,150	0	80,000	0	88,750
Lebanon, TR-8B				14,000		28,500	42,500
Lebanon, Total	7,716,126	3,600	5,150	14,000	80,000	28,500	131,250
*Lempster	237,479				3,900		3,900
Lisbon, TR-4A		0	0	0	0	0	0
Lisbon, TR-4B		1,600	545	200	13,700		16,045
*Lisbon, Gale River					2,500		2,500
Lisbon, Total	2,363,412	1,600	545	200	16,200	0	18,545
Littleton	4,915,309	4,600	3,050	10,500	7,400	0	25,550
*Lyman	330,915				3,100		3,100
Lyme	794,183	0	1,650	0	280,900	0	282,550
*Marlboro	1,251,100	200	700	7,400	11,700		20,000
*Marlow	285,929			700	6,000		6,700
*Milan	597,396				5,100		5,100
Monroe	9,066,391	0	200	0	200		400
*Nelson	358,995				5,400		5,400
*New London	1,903,627				16,000		16,000
Newport	4,608,470	0	1,125	0	26,400	4,600	32,125
*Northumberland	2,483,504	2,700	2,110	27,730	3,100	2,300	42,990
*Orange	125,045				1,000		1,000
Orford	742,656	2,050	3,010	0	166,300	0	171,360
Piermont	667,290	0	5,760	300	14,900	0	20,960
*Pittsburg	2,566,702		500	1,000	3,000		4,500
Plainfield	761,415	0	3,030	0	11,900	0	14,930
*Richmond	286,249				10,500		10,500
*Roxbury	122,182				1,800		1,800
*Springfield	419,557				8,000		8,000
*Stark	342,908				16,300		16,300
*Stewartstown	863,720		2,530	100	800	1,500	4,930
*Stratford	1,067,990		325	9,500	2,500	5,000	17,325
*Sullivan	183,932				5,100		5,100
*Sunapee	2,143,580	400			2,500	1,100	4,000
*Surry	362,547				1,600		1,600
Swanzey	1,534,938	1,000		29,750	7,000		37,750
*Troy	1,049,614			2,000	8,000		10,000

TABLE 19
SHEET 3 of 3

Town (1)	1935 Assessed Value (2)	Direct Flood Loss: Urban (3)	Rural (4)	Industrial (5)	Highway (6)	Railroad (7)	Total (8)
*Unity	$ 360,290	$	$	$	$ 300	$	$ 300
Walpole, C-4		1,500	0	15,375	0	0	16,875
Walpole, C-5		0	21,495	0	90,000	17,200	128,695
Walpole, Total	3,518,165	1,500	21,495	15,375	90,000	17,200	145,570
Westmoreland	552,795	0	14,995	400	15,500	0	30,395
*Whitefield	2,040,409				3,300		3,300
Winchester, TR-14F					14,000	2,300	16,300
Winchester, TR-14G		7,545	2,650	70,200			30,395
Winchester, Total	2,133,310	7,545	2,650	70,200	14,000	2,300	96,695
Total(N.H.Towns)	129,337,275	56,395	162,690	537,537	1,375,250	185,600	2,317,472
*Estimate of losses not included above		105	110	63	50	24,200	24,528
GRAND TOTAL New Hampshire		56,500	162,800	537,600	1,375,300	209,800	2,342,000
		(56,000	163,000	538,000	1,375,000	210,000	2,342,000
20-Reservoir Plan:							
Below Reservoirs	80,353,296	51,095	150,065	459,717	992,950	123,800	1,777,627
Above "		4,905	12,935	78,283	382,050	86,200	564,373

Assessed value from Report N. H. Tax Commission for year of 1935, which includes all assessable property, such as real estate, live stock, furniture, machinery, automobiles, etc. Total for towns.

TABLE 20
SHEET 1 OF 3
DIRECT FLOOD LOSSES - CONNECTICUT RIVER WATERSHED - 1936 FLOOD
STATE OF MASSACHUSETTS
SUMMARY OF DIRECT LOSSES AND ASSESSED VALUATIONS
OF TOWNS REPORTING LOSSES.

(NOTE: * IDENTIFIES LOSSES NOT SUBJECT TO CONTROL BY STUDIED RESERVOIR PLANS. NUMBER IN COLUMN (1) REFERS TO DAMAGE ZONES.)

TOWN	1935 ASSESSED VALUE	DIRECT FLOOD LOSS URBAN	RURAL	INDUSTRIAL	HIGHWAY	RAILROAD	TOTAL
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
AGAWAM	$9,736,254	$34,650	$ 5,000	$ 10,000	$33,900	$ 2,200	$ 85,750
*AMHERST	10,144,491	-	-	-	5,000	-	5,000
*ASHBURNHAM	1,764,207	1,000	-	-	33,000	-	34,000
ATHOL #15E		0	0	268,500	20,000	55,900	344,400
ATHOL #15G		118,300	7,775	75,000	63,800	46,000	310,875
ATHOL #15F		0	0	0	27,000	0	27,000
ATHOL, TOTAL	11,806,947	118,300	7,775	343,500	110,800	101,900	682,275
*BARRE	3,186,361	-	-	40,000	154,700	3,000	197,700
*BELCHERTOWN	1,573,920	0	2,000	0	15,400	-	17,400
*BERNARDSTON	956,704	-	1,800	-	-	-	1,800
*BRIMFIELD	963,058	-	-	-	2,500	2,100	4,600
*BROOKFIELD	1,417,098	2,500	-	2,000	4,700	2,400	11,600
*BUCKLAND	3,096,637	-	2,000	5,900	110,000	5,700	123,600
*CHARLEMONT	1,204,352	1,000	4,000	200	42,000	2,300	49,500
*CHESTER	1,458,554	4,000	7,000	2,800	17,000	1,700	32,500
*CHESTERFIELD	680,450	-	-	-	1,000	-	1,000
CHICOPEE, C-8		333,600	0	1,240,000	139,000	29,800	1,742,400
*CHICOPEE, #17B		0	3,200	26,000	2,000	0	31,200
CHICOPEE, TOTAL	42,446,529	333,600	3,200	1,266,000	141,000	29,800	1,773,600
*COLRAIN	1,548,080	-	-	7,000	9,000	-	16,000
*CONWAY	1,007,778	-	400	-	5,800	11,400	17,600
*CUMMINGTON	557,488	3,000	-	300	4,700	-	8,000
DEERFIELD C-7		11,600	34,800	13,000	8,000	-	67,400
*DEERFIELD					12,000	3,400	15,400
DEERFIELD, TOTAL	4,083,436	11,600	34,800	13,000	20,000	3,400	82,800
*EAST BROOKFIELD	1,159,871	0	500	1,900	5,000	-	7,400
EASTHAMPTON	10,497,268	0	0	38,000	11,000	2,800	51,800
ERVING #15G		5,100	0	130,000	48,850	28,700	212,650
*ERVING		-	-	-	5,000	-	5,000
ERVING, TOTAL	2,251,699	5,100	0	130,000	53,850	28,700	217,650
*GARDNER	24,071,973	0	4,400	2,800	27,000	0	34,200
GILL	935,708	0	17,600	1,000	6,360	0	24,960
*GRANBY	1,005,790	-	-	-	2,525	-	2,525
*GRANVILLE	2,015,693	-	1,000	-	17,000	-	18,000
GREENFIELD		31,400	-	45,600	809,800	-	886,800
*GREENFIELD					30,000	1,100	31,100
GREENFIELD, TOTAL	29,813,607	31,400	-	45,600	839,800	1,100	917,900
HADLEY	3,028,755	182,200	154,200	10,000	61,000	17,200	424,600
*HARDWICK	1,833,293	0	4,800	91,100	5,700	3,500	105,100
HATFIELD	2,731,693	50,500	162,660	180,000	108,660	-	501,820
*HAWLEY	250,033	1,000	-	-	8,500	0	9,500
HOLYOKE	90,893,212	147,900	1,170	800,000	70,000	92,230	1,111,300
*HUBBARDSTON	781,981	700	-	-	12,400	200	13,300

Town	1935 Assessed Value	Direct Flood Loss: Urban	Rural	Indus-trial	Highway	Railroad	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Huntington #18	$	$ 2,000	$ 0	$ 20,300	$ 18,700	$10,700	$ 51,700
*Huntington		-	7,300	10,000	-	-	17,300
Huntington, Total	1,013,236	2,000	7,300	30,300	18,700	10,700	69,000
*Leverett	506,057	-	2,000	500	3,000	-	5,500
Longmeadow	13,105,622	9,100	16,900	0	700	-	26,700
*Ludlow	8,531,062	-	800	5,000	39,500	2,400	47,700
*Monroe	1,262,899	-	-	2,000	1,500	-	3,500
*Monson	3,590,617	-	-	1,000	1,500	-	2,500
Montague #15		0	0	0		22,900	22,900
Montague C-7		13,000	8,500	140,000	65,000	81,330	307,830
Montague, Total	10,397,227	13,000	8,500	140,000	65,000	104,230	330,730
Montgomery	301,711	-	-	-	-	-	0
*New Braintree	522,926	-	-	-	-	8,600	8,600
*New Salem	476,257	-	-	-	17,600	-	17,600
Northampton	28,352,152	235,600	43,700	238,000	94,700	5,570	622,570
Northfield	2,044,050	-	246,200	3,900	126,200	182,000	558,300
Orange	5,257,129	25,000	2,025	315,000	82,900	57,200	482,125
*Otis	592,621	-	-	-	2,000	-	2,000
*Palmer	8,564,981	600	1,700	50,500	269,400	5,500	327,700
*Pelham	753,185	-	-	-	7,500	-	7,500
*Peru	312,590	-	-	-	2,000	-	2,000
*Phillipston	401,220	-	-	-	18,000	-	18,000
*Rowe	776,432	-	-	-	-	3,400	3,400
Royalston	856,710	2,250	3,160	21,000	122,000	28,600	177,010
Russell	4,464,829	4,900	0	13,300	11,100	20,300	49,600
*Rutland	1,352,257	-	-	-	37,000	1,100	38,100
*Sandisfield	701,124	0	500	180	2,500	-	3,180
*Shelburne	3,021,212	2,500	-	4,000	31,000	-	37,500
*Shutesbury	453,636	-	-	-	3,900	-	3,900
So. Hadley	9,033,148	250,100	3,000	36,000	78,000	-	367,100
*Southampton	1,006,746	-	-	-	775	-	775
*Southwick	2,250,633	-	1,000	3,000	2,500	-	6,500
*Spencer	4,539,024	1,250	2,000	5,000	55,300	2,600	66,150
*Springfield #17b		-	-	-	1,000	-	1,000
Springfield C-8		1,574,200	0	1,370,000	315,000	8,100	3,267,300
Springfield, Total	306,672,839	1574,200	0	1,370,000	316,000	8,100	3,268,300
Sunderland	1,210,785	9,500	42,450	1,100	807,700	-	860,750
*Templeton	3,306,220	-	-	-	49,000	1,000	50,000
*Tolland	402,469	-	400	-	500	-	900
*Ware	5,421,078	10,000	2,200	6,000	133,300	1,400	152,900
*Warren	2,599,728	0	150	255,000	34,000	8,500	297,650

TABLE 20
SHEET 3 of 3

Town	1935 Assessed Value	Direct Flood Loss: Urban	Rural	Industrial	Highway	Railroad	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
*Warwick	$382,963	-	-	-	$14,500	-	$14,500
Wendell	1,014,141	200	2,500	140,500	10,000	29,700	182,900
*W. Brookfield	1,470,146	300	1,000	0	12,200	3,300	16,800
Westfield	19,874,158	71,500	17,000	78,500	25,000	10,000	202,000
*Westhampton	411,400	-	-	-	3,000	-	3,000
W.Springfield C-8		1,521,600	83,000	1,250,000	50,000	131,000	3,035,600
W.Springfield #18		-	-	-	10,000	-	10,000
W.Springfield, Total	26,244,480	1,521,600	83,000	1,250,000	60,000	131,000	3,045,600
Whately	1,153,381	-	29,460	-	14,500	-	43,960
*Wilbraham	3,109,577	1,250	1,000	5,200	34,500	-	41,950
Winchendon #15C		188,200	-	286,000	166,500	9,400	650,100
*Winchendon		-	-	-	25,975	-	25,975
Winchendon, Total	5,741,929	188,200	-	286,000	192,475	9,400	676,075
*Windsor	504,895	-	-	-	3,800	-	3,800
Total (Mass. Towns)	762,869,952	4,851,500	937,250	7,252,080	4,744,045	946,230	18,731,105
Estimate of losses not included above,		96,500	17,750	113,920	29,955	10,770	263,395
GRAND TOTAL, Massachusetts,		$4,948,000	955,000	7,366,000	4,774,000	957,000	19,000,000
20-Reservoir Plan							
Below Reservoirs	644,963,135	4,822,400	886,100	6,724,700	3,405,370	871,630	16,710,200
Above "		125,600	68,900	641,300	1,368,630	85,370	2,289,800

Assessed Valuations from Manual of the General Court, 1935-1936 (State of Massachusetts)

TABLE 21
DIRECT FLOOD LOSSES - CONNECTICUT RIVER WATERSHED - 1936 FLOOD
STATE OF CONNECTICUT
SUMMARY OF DIRECT LOSSES AND ASSESSED VALUATIONS
OF TOWNS REPORTING LOSSES

(NOTE: *IDENTIFIES LOSSES NOT SUBJECT TO CONTROL BY STUDIED RESERVOIR PLANS. NUMBER IN COLUMN (1) REFERS TO DAMAGE ZONES.)

Town	Assessment 1935 Grand List Value	Direct Losses: Urban	Rural	Industrial	Highway	Railroad	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
*Avon	$ 3,378,731	$ 0	$ 1,850	$ 0	$ 0	$ 0	$ 1,850
*Barkhamsted	1,140,873	1,000	0	2,950	1,150	0	5,100
*Bristol	51,064,425	0	0	0	600	0	600
*Burlington	989,465	500	0	0	0	0	500
*Canton	3,373,176	0	0	400	850	0	1,250
Chester	1,403,003	0	1,330	0	1,150	1,500	3,980
*Colebrook	1,289,975	0	0	0	3,315	0	3,315
Cromwell	3,703,494	32,390	60,290	21,916	12,480	1,500	128,576
*East Granby	1,789,043	0	0	0	150	1,100	1,250
East Haddam	2,874,080	3,590	18,200	14,000	5,200	0	40,990
East Hampton	3,844,268	0	400	0	2,550	0	2,950
East Hartford	35,659,887	750,500	31,800	511,900	47,000	15,500	1,356,700
East Windsor	4,156,915	41,040	9,260	9,300	6,600	0	66,200
Enfield	19,374,633	3,100	1,500	5,000	8,775	0	18,375
Essex	4,098,693	2,100	0	43,000	0	0	45,100
*Farmington	7,914,004	1,600	2,100	5,900	2,800	9,100	21,500
Glastonbury	8,846,911	137,600	23,670	30,791	10,320	0	202,381
*Granby	1,554,693	0	0	0	2,700	0	2,700
Haddam	1,883,090	0	17,395	0	550	1,500	19,445
Hartford	352,319,419	2,222,000	23,000	4,895,300	450,000	70,000	7,660,300
*Hartland	593,296	0	500	0	500	0	1,000
Lyme	1,526,274	6,720	0	0	400	0	7,120
Middletown	34,215,609	121,800	0	218,500	179,000	5,700	525,000
*New Hartford	2,442,408	60,000	0	140,000	27,000	0	227,000
Portland	5,955,769	38,000	27,555	151,900	29,100	800	247,355
Rocky Hill	3,070,772	0	1,300	37,800	300	1,500	40,900
Saybrook	2,603,123	50	0	100	1,680	0	1,830
*Simsbury	8,133,916	1,000	1,200	0	2,300	0	4,500
S. Windsor, C-9		0	15,555	0	0	0	15,555
S. Windsor, C-10		14,160	76,260	500	15,450	0	106,370
S. Windsor, Total	3,539,869	14,160	91,815	500	15,450	0	121,925
Suffield	7,297,497	0	610	0	0	2,170	2,780
Wethersfield	12,329,078	16,240	5,850	81,450	15,310	8,300	127,150
*Winchester	14,074,684	0	0	500	35,700	0	36,200
*Windsor		0	0	4,000	0	0	4,000
Windsor, C-9		0	9,275	0	15,980	2,480	27,735
Windsor, C-10		27,200	26,700	104,463	0	2,500	160,863
Windsor, Total	14,440,687	27,200	35,975	108,463	15,980	4,980	192,598
Windsor Locks	5,608,343	3,400	0	185,200	1,935	3,250	193,785
Total (Conn. Towns)	626,490,103	3,483,990	355,600	6,464,870	880,845	126,900	11,312,205
*Estimate of Losses not included above:		56,010	20,400	4,130	155	100	80,795
Grand Total	626,490,103	3,540,000	376,000	6,469,000	881,000	127,000	11,393,000
20-Reservoir Plan:							
Below Reservoirs:	528,751,414	3,419,890	349,950	6,311,120	803,780	116,700	11,001,440
*Above Reservoirs:	97,738,689	120,110	26,050	157,880	77,220	10,300	391,560

1935 GRAND LIST VALUES FROM "CONNECTICUT STATE REGISTER AND MANUAL - 1936"

TABLE 22

DIRECT LOSSES - CONNECTICUT RIVER WATERSHED

SUMMARY OF 1936 LOSSES BY STATES

	1936 Direct Flood Losses in Thousands of Dollars Estimate						
State	Urban	Rural	Industrial	Highway	Railroad	Total	Percent
Vermont	101	183	337	691	453	1,765	5.1
New Hampshire	56	163	538	1,375	210	2,342	6.8
Massachusetts	4,948	955	7,366	4,774	957	19,000	55.1
Connecticut	3,540	376	6,469	881	127	11,393	33.0
Totals	8,645	1,677	14,710	7,721	1,747	34,500	100.0
Percent	25.0	4.9	42.6	22.4	5.1	100.0	

TABLE 23 3/15/37

DIRECT LOSSES - CONNECTICUT RIVER WATERSHED

SUMMARY OF 1936 LOSSES BY RIVER BASINS
(Not limited to losses below reservoirs)

RIVER BASIN		STATE	Damages (1936) (In thousands of dollars)					
			Urban	Rural	Inds-trial	Railroad	Highway	Total
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)
Connecticut	(1)	Conn. Mass. N.H. & Vt.	7,916	1,508	12,066	1,129	4,914	27,533
Israel	*	N.H.				1	23	24
Passumpsic		Vt.	12	6	38	10	6	72
Ammonoosuc		N.H.	6	10	20	4	95	135
Stevens		Vt.					3	3
Wells	(2)	Vt.	2		3			5
Waits		Vt.					4	4
Ompompanoosuc		Vt.					1	1
White	(3)	Vt.		3	2	24	13	42
Ottauquechee		Vt.	1		9		5	15
Black		Vt.	9	1	12	5	47	74
Williams	*	Vt.					10	10
Saxtons		Vt.					15	15
West	(4)	Vt.		3	9		127	139
Westfield		Mass.	85	32	128	43	96	384
Farmington		Conn. Mass.	64	8	154	10	99	335
Total			8,095	1,571	12,441	1,226	5,458	28,791
Per Cent of Total			28.1	5.4	43.2	4.3	19.0	100.0
Other Rivers								
Mascoma	*	N.H.	1		18	40	36	95
Sugar		N.H.	2	1	23	9	108	143
Ashuelot		N.H.	31	15	355	9	126	536
Cold River	*	N.H.		1			35	36
Millers	(5)	Mass. N.H.	340	20	1,239	279	719	2,597
Deerfield	*	Vt. Mass.	4	8	19	92	317	440
Chicopee		Mass.	17	19	489	45	849	1,419
Misc., Other Streams	*	Various	155	42	126	47	73	443
Total, Other Rivers			550	106	2,269	521	2,263	5,709
GRAND TOTAL			8,645	1,677	14,710	1,747	7,721	34,500
Per Cent of Total			25.0	4.9	42.6	5.1	22.4	100.0

* No detailed investigation for 1936 stage-loss relationship.
(1) Exclusive of tributaries listed in table. There were 6 lives lost.
(2) There was one life lost in Wells River Basin.
(3) There were two lives lost in White River Basin.
(4) There was one life lost in West River Basin.
(5) There was one life lost in Millers River Basin.

TABLE 24

DAMAGE ZONES FOR CONNECTICUT RIVER AND TRIBUTARIES

Connecticut River.

1. From Fifteen Mile Falls through Towns of Newbury, Vermont, and Haverhill, New Hampshire.
2. From southern tomwship lines of Newbury, Vermont, and Haverhill, New Hampshire, to Wilder tailwater.
3. From Wilder Dam through Towns of Windsor, Vermont, and Cornish, New Hampshire.
4. From southern township lines of Windsor, Vermont, and Cornish, New Hampshire, to Bellows Falls Dam.
5. From Bellows Falls Dam to Vernon Dam.
6. From Vernon Dam to mouth of Millers River.
7. From mouth of Millers River through Towns of Holyoke and South Hadley, Massachusetts.
8. From southern township lines of Holyoke and South Hadley to Massachusetts-Connecticut state line.
9. From Massachusetts-Connecticut state line through mouth of Farmington River.
10. Below mouth of Farmington River.

Tributaries.

1-f. Passumpsic River from East Haven site to Twin State Gas and Electric Co. Dam No. 1-1/2 at St. Johnsbury.

1-d. Moose River below Victory site.

1-e. Passumpsic River below Twin State Gas and Electric Co. Dam No. 1-1/2 at St. Johnsbury.

2. Stevens River below Harvey Lake.

3. Wells River below Groton Pond.

4-a. Ammonoosuc River from Bethlehem Junction dam site to mouth of Gale River.

4-b. Ammonoosuc River below mouth of Gale River.

5. Waite River bleow south Branch dam site.

7-a. White River from Gaysville site through Bethel.

7-e. Third Branch below Randolph site.

TABLE 24

Tributaries (Continued).

7-b. White River from Bethel through Town of South Royalton.

7-c. White River below South Royalton.

8-a. Mascoma River from West Canaan site to Mascoma Lake.

8-b. Mascoma River below Mascoma Lake.

9. Ottauquechee River bloow Bridgwater Corners dam site.

10-a. Sugar River from Croydon dam site to Claremont dam site.

10-b. Sugar River below Claremont dam site.

11-a. Black River from Ludlow dam site to North Springfield dam site.

11-b. Black River below North Springfield dam site.

12. Saxtons River below Cambridgeport dam site.

13. West River belwo Newfane dam site.

13-X. West River above Newfane dam site.

14-g. Ashuelot River from Surry Mountain site through Village of Ashuelot.

14-f. Ashuelot River below Village of Ashuelot.

15-c. Millers River from Lower Naukeag site at Birch Hill site.

15-e. Millers River from Birch Hill site to Starrett Dam at Athol.

15-f. Tully River below Tully Dam site.

15-g. Millers River from Starrett Dam in Athol to U.S.G.S. gaging station at Erving.

15-h. Millers River below U.S.G.S. gaging station at Erving.

16-a. Deerfield River in Vermont.

16-b. Deerfield River in Massachusetts.

17-a. Swift River below Quabbin Reservoir.

17-b. Chicopee River.

18. Westfield River below Knightville dam site.

19-a. Farmington River in Massachusetts.

19-b. Farmington River from Massachusetts-Connecticut state line to town of Avon.

19-c. Farmington River from above Town of Avon to mouth.

21. Quaboag River.

22. Ware River.

TABLE 25
DIRECT FLOOD LOSSES - CONNECTICUT RIVER WATERSHED
SUMMARY OF RECURRING LOSSES BELOW CONSIDERED RESERVOIR SITES BASED UPON 1936 FLOOD LOSSES

RIVER (1)		ZONE (2)		DIRECT FLOOD LOSS URBAN (3)	RURAL (4)	INDUSTRIAL (5)	HIGHWAY (6)	RAILROAD (7)	TOTAL (8)
ONNECTICUT	CONN.	10	(1)	$ 3,372,400	$ 313,700	$ 6,111,600	$ 770,500	$ 108,800	$10,677,000
	"	9	(1)	47,500	36,200	199,500	33,300	7,900	324,400
	MASS.	8	(1)	3,473,200	104,900	3,870,000	538,600	171,100	8,157,800
	"	7	(1)	931,800	484,900	1,701,700	2,128,400	199,100	5,445,900
MASS, VT., N.H.		6	(1)		269,900	5,900	150,100	191,400	617,300
VT., N.H.		5	(1)	9,600	140,700	192,000	364,500	298,000	1,004,800
" "		4	(1)	1,500	64,900	15,400	182,800	43,600	308,200
" "		3	(1)	47,900	29,300	87,600	111,000	34,400	310,200
" "		2	(1)	15,300	22,000	8,700	563,800	38,000	647,800
" "		1	(1)	13,300	28,200	12,200	24,400	26,500	104,600
OTAL FOR CONNECTICUT RIVER				$7,912,500	$1,494,700	$12,204,600	$4,867,400	1,118,800	$27,598,000
RIBUTARY STREAMS									
PASSUMPSIC & MOOSE, VT.		1-F	(1)	6,700	3,300	7,800	3,500	800	22,100
" " " "		1-B	(1)	-	-	-	2,500	2,000	4,500
" " " "		1-E	(1)	5,700	2,800	25,000	-	7,000	40,500
STEVENS, VT.		2	(1)	-	-	-	3,000	-	3,000
WELLS, VT.		3	(1)	1,600	-	3,200	-	-	4,800
AMMONOOSUC, N.H.		4-A	(1)	4,600	4,100	10,500	9,700	-	28,900
" "		4-B	(1)	1,600	6,000	9,200	14,700	2,900	34,400
WAITS, VT.		5	(1)	200	-	-	-	-	200
WHITE, VT.		7-A	(1)	-	-	-	8,000	-	8,000
" "		7-E	(1)	-	-	-	-	-	0
" "		7-B	(1)	100	200	-	4,000	-	4,300
" "		7-C	(1)	300	2,100	1,600	1,300	24,000	29,300
MASCOMA, N. H.		8-A	*	500	-	2,200	3,700	11,800	18,200
" "		8-B	*	-	-	14,000	-	28,500	42,500
OTTAUQUECHEE, VT.		9		700	-	8,200	3,200	-	12,100
SUGAR, N. H.		10-A		-	1,400	2,000	26,400	8,000	37,800
" "		10-B	(1)	1,300	0	20,500	29,600	0	51,400
BLACK, VT.		11-A		2,500	300	9,500	9,300	5,000	26,600
" "		11-B	(1)	6,500	600	2,800	30,600	-	40,500
SAXTONS, VT.		12		-	-	-	3,300	-	3,300
WEST, VT.		13	(1)	-	2,000	4,000	25,000	-	31,600
ASHUELOT, N.H.		14-G	(1)	26,800	6,500	191,200	21,000	1,100	246,600
" "		14-F	(1)	2,900	7,900	149,800	25,000	8,000	193,600
MILLERS, MASS.		15-C	(1)	188,200	-	286,000	166,500	9,400	650,100
" "		15-E	(1)	2,200	3,200	289,500	142,000	84,500	521,400
" "		15-F	(1)	-	-	-	27,000	-	27,000
" "		15-G	(1)	148,600	12,300	610,500	168,900	132,900	1,073,200
" "		15-H	(1)	-	-	50,000	36,600	51,600	138,200
DEERFIELD, VT.		16-A	*	-	1,500	-	66,800	64,400	132,700
" MASS.		16-B	*	3,500	6,400	19,100	216,300	27,300	272,600
SWIFT & CHICOPEE, MASS.		17-A & B		0	8,700	61,000	39,100	2,400	111,200
WESTFIELD, MASS.		18	(1)	78,400	17,000	112,100	64,800	41,000	313,300
FARMINGTON, MASS.		19-A		0	900	200	500	0	1,600
" CONN.		19-B		12,000	500	83,400	22,300	0	118,200
" "		19-C		2,600	5,200	9,900	5,300	10,200	33,200
QUABOAG, MASS.		21		3,400	3,700	19,900	50,600	18,900	96,500
WARE, MASS.		22		10,000	7,000	156,600	90,100	22,000	285,700
OTAL FOR TRIBUTARY STREAMS				$510,900	$104,200	$2,159,700	$1,320,600	$563,700	$4,659,100
RAND TOTAL				$8,423,400	$1,598,900	$14,364,300	$6,188,000	1,682,500	$32,257,100
)-RESERVOIR PLAN (1)									
CONNECTICUT RIVER ZONE TOTAL				7,912,500	1,494,700	12,204,600	4,867,400	1,118,800	27,598,000
TRIBUTARY STREAMS ZONE TOTAL				475,700	68,600	1,773,700	783,700	365,200	3,466,900
OTAL				8,388,200	1,563,300	13,978,300	5,651,100	1,484,000	31,064,900
ONES OUTSIDE 20-RESERVOIR PLAN				35,200	35,600	386,000	536,900	198,500	1,192,200

NOTE: * INDICATES 1936 FLOOD LOSSES; NO DETAILED INVESTIGATION.
(1) INDICATES ZONES AFFECTED UNDER THE 20-RESERVOIR PLAN.

TABLE 26

COMPARISON OF 1936 DIRECT FLOOD LOSSES AND PROPERTY VALUE DEPRECIATION
MAJOR CITIES IN MASSACHUSETTS AND CONNECTICUT

Location	1930 Population	Total Area Acres	1936 Total Assessed Valuation	Flooded Area Acres	Estimated Assessed Valuation Flooded Area	Total Direct 1936 Flood Loss	Depreciation of Property Values
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Hartford, Conn.	164,072	11,158	$352,319,419*	3,045	$140,000,000	$ 7,660,300	$35,000,000
East Hartford, Conn.	17,125	11,674	35,659,887*	2,900	10,398,150	1,356,700	2,600,000
Springfield, Mass.	149,900	20,288	277,952,555	910	73,959,150	3,268,300	18,500,000
West Springfield, Mass.	16,684	10,752	26,244,480*	1,800	18,278,000	3,045,600	4,570,000
Chicopee, Mass.	43,930	14,656	41,798,440	1,448	11,664,100	1,773,600	2,900,000
Holyoke, Mass.	56,537	13,568	83,527,180	460	14,086,600	1,111,300	4,000,000
Northampton, Mass.	24,381	22,144	26,032,800	3,500	2,870,890	622,570	432,000
TOTALS	472,629	104,240	$843,534,761	14,063	$271,256,890	$18,838,370	$68,002,000

* 1935 Assessed Valuation

TABLE 27

FLOOD LOSSES BELOW CONSIDERED RESERVOIR SITES - CONNECTICUT RIVER WATERSHED
ESTIMATED DIRECT AND INDIRECT LOSSES FOR 1936 FLOOD AND
DEPRECIATION OF PROPERTY VALUES BECAUSE OF FLOODS

River		Damage Zone	Recurring Flood Losses: Direct	Recurring Flood Losses: Indirect	Est. Depreciation of Property Values
(1)		(2)	(3)	(4)	(5)
Connecticut	Conn.	10 (1)	$10,677,000	$11,304,650	$38,819,000
	"	9 (1)	324,400	307,350	453,000
	Mass.	8 (1)	8,157,800	8,770,800	25,970,000
	"	7 (1)	5,445,900	4,254,300	5,341,000
	Mass., Vt., N.H.	6 (1)	617,300	242,750	81,000
	Vt., N. H.	5 (1)	1,004,800	634,750	540,000
	" "	4 (1)	308,200	147,700	181,000
	" "	3 (1)	310,200	237,000	230,000
	" "	2 (1)	647,800	338,200	53,000
	" "	1 (1)	104,600	62,600	54,000
Total for Connecticut River			$27,598,000	$26,300,100	$71,722,000
Tributary Streams					
Passumpsic & Moose, Vt.		1-f (1)	22,100	19,150	126,000
" " " "		1-d (1)	4,500	2,650	17,000
" " " "		1-e (1)	40,500	40,200	22,000
Stevens, Vt.		2 (1)	3,000	1,500	1,000
Wells, Vt.		3 (1)	4,800	5,500	6,500
Ammonoosuc, N. H.		4-a (1)	28,900	22,450	12,000
" "		4-b (1)	34,400	22,250	10,000
Waits, Vt.		5 (1)	200	200	2,500
White, Vt.		7-a (1)	8,000	4,000	30,000
" "		7-e (1)	0	0	24,000
" "		7-b (1)	4,300	2,100	10,000
" "		7-c (1)	29,300	19,850	21,000
Mascoma, N. H.		8-a *	18,200	13,250	5,000
" "		8-b *	42,500	36,000	70,000
Ottauquechee, Vt.		9	12,100	11,700	50,000
Sugar, N. H.		10-a	37,800	21,200	8,000
" "		10-b (1)	51,400	39,700	90,000
Black, Vt.		11-a	26,600	21,850	37,500
" "		11-b (1)	40,500	26,000	20,000
Saxtons, Vt.		12	3,300	1,650	8,000
West, Vt.		13 (1)	31,600	17,400	28,000
Ashuelot, N. H.		14-g (1)	246,600	260,500	535,000
" "		14-f (1)	193,600	193,000	60,000
Millers, Mass.		15-c (1)	650,100	630,450	200,000
" "		15-e (1)	521,400	463,000	420,000
" "		15-f (1)	27,000	13,500	0
" "		15-g (1)	1,073,200	1,044,050	730,000
" "		15-h (1)	133,200	111,400	170,000

TABLE 27 (Cont.)

River	Damage Zone	Recurring Flood Losses Direct	Recurring Flood Losses Indirect	Est. Depreciation of Property Values
(1)	(2)	(3)	(4)	(5)
Deerfield, Vt.	16-a *	132,700	78,700	67,000
" , Mass.	16-b *	272,600	153,650	338,000
Swift, Mass.	17-a	41,100	38,950	130,000
Chicopee, Mass.	17-b	70,100	52,700	650,000
Westfield, Mass.	18 (1)	313,300	280,000	600,000
Farmington, Mass.	19-a	1,600	550	10,000
" , Conn.	19-b	118,200	120,050	500,000
" "	19-c	33,200	24,550	5,000
Quaboag, Mass.	21	96,500	65,500	100,000
Ware, Mass.	22	285,700	251,050	120,000
Total for Tributary Streams		$4,659,100	$4,110,200	$5,233,500
GRAND TOTAL		$32,257,100	$30,410,300	$76,955,500

20-Reservoir Plan (1)

Connecticut River Zone Total	$27,598,000	$26,300,100	$71,722,000
Tributary Streams Zone Total	3,466,900	3,218,850	3,135,000
Total	$31,064,900	$29,518,950	$74,857,000
Zones outside 20-Reservoir Plan	1,192,200	891,350	2,098,500

Note.- * Indicates 1936 Flood Losses, No Detailed Investigation.
(1) Indicates Zones Affected Under the 20-Reservoir Plan.

TABLE 28

1936 FLOOD - CONNECTICUT RIVER WATERSHED
STATEMENT SHOWING AREA FLOODED AND DAMAGE TO AGRICULTURAL LAND

State	Total Area Flooded (Excl. of Normal River Area)	Agricultural Land				
		Acres Flooded	Acres Damaged by			Estimated Damage
			Erosion	Heavy Deposits	Light Deposits	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Vermont	8,980	6,700	133	373	2,065	$63,900
New Hampshire	8,130	6,830	331	858	1,715	71,275
Massachusetts	18,730	12,890	1,086	953	1,091	327,210
Connecticut	27,550	8,100	66	346	1,282	53,125
Total	63,390	34,520	1,616	3,030	6,153	$515,510

Note: Estimated damage in column (7) is damage sustained by reason of erosion and silting only; it does not include losses to buildings, crops, livestock, etc.

TABLE 29
Estimate of Depreciation of Property Values in Flooded Towns, Flood of 1936, Connecticut River Watershed. Twenty-Reservoir Plan.

River	Zone	Total Population: 1930 Census	Total Pre-Flood Assessed Valuation	Est. Value of Property in Flooded Areas	Est. Depreciation of Property Values
(1)	(2)	(3)	(4)	(5)	(6)
Connecticut	10	250,264	$ 434,103,339	$155,441,150	$ 38,819,000
	9	30,638	44,648,075	5,830,000	453,000
	8	216,946	397,231,774	104,381,200	25,970,000
	7	132,630	191,200,164	25,456,900	5,341,000
	6	3,505	3,391,885	717,000	31,000
	5	16,780	19,324,815	4,794,000	540,000
	4	6,750	11,662,440	1,420,000	181,000
	3	10,331	10,639,945	3,227,000	230,000
	2	8,728	11,185,628	570,000	53,000
	1	7,702	15,711,821	452,000	54,000
Total for Connecticut River		684,824	$1,139,649,886	$302,239,250	$71,722,000
Tributary Streams					
Passumpsic, Vt.	1f	7,471	5,711,133	1,230,000	126,000
& Moose, Vt.	1d	2,234	2,098,673	245,000	17,000
"	1e	5,094	4,825,676	400,000	22,000
Stevens, Vt.	2	260	266,000	20,000	1,000
Wells, Vt.	3	1,869	1,459,648	65,000	6,500
Ammonoosuc, N.H.	4a	5,830	8,652,448	260,000	12,000
"	4b	2,274	3,152,412	200,000	10,000
Waits, Vt.	5	370	331,000	50,000	2,500
White, Vt.	7a	2,110	1,359,739	550,000	24,000
"	7e	1,957	2,401,640	240,000	30,000
"	7b	1,491	848,281	100,000	10,000
"	7c	2,747	2,793,288	420,000	21,000
Sugar, N.H.	10b	9,377	11,193,480	2,160,000	90,000
Black, Vt.	11b	4,443	8,494,052	200,000	20,000
West, Vt.	13	1,832	1,553,350	230,000	23,000
Ashuelot, N.H.	14g	17,593	21,152,052	5,350,000	535,000
"	14f	1,357	3,140,375	600,000	60,000
Millers, Mass.	15c	6,202	5,741,929	2,000,000	200,000
"	15e	6,094	6,760,183	4,200,000	420,000
"	15f	1,050	1,180,695	300,000	0
"	15g	10,628	12,119,399	7,300,000	730,000
"	15h	630	1,125,849	500,000	170,000
Westfield, Mass	18	23,095	26,627,934	1,907,000	600,000
Total for Tributaries		117,058	$132,990,286	$ 28,527,000	$3,135,000
Total (20-Reservoir Plan)		801,882	$1,322,640,172	$330,816,250	$74,357,000

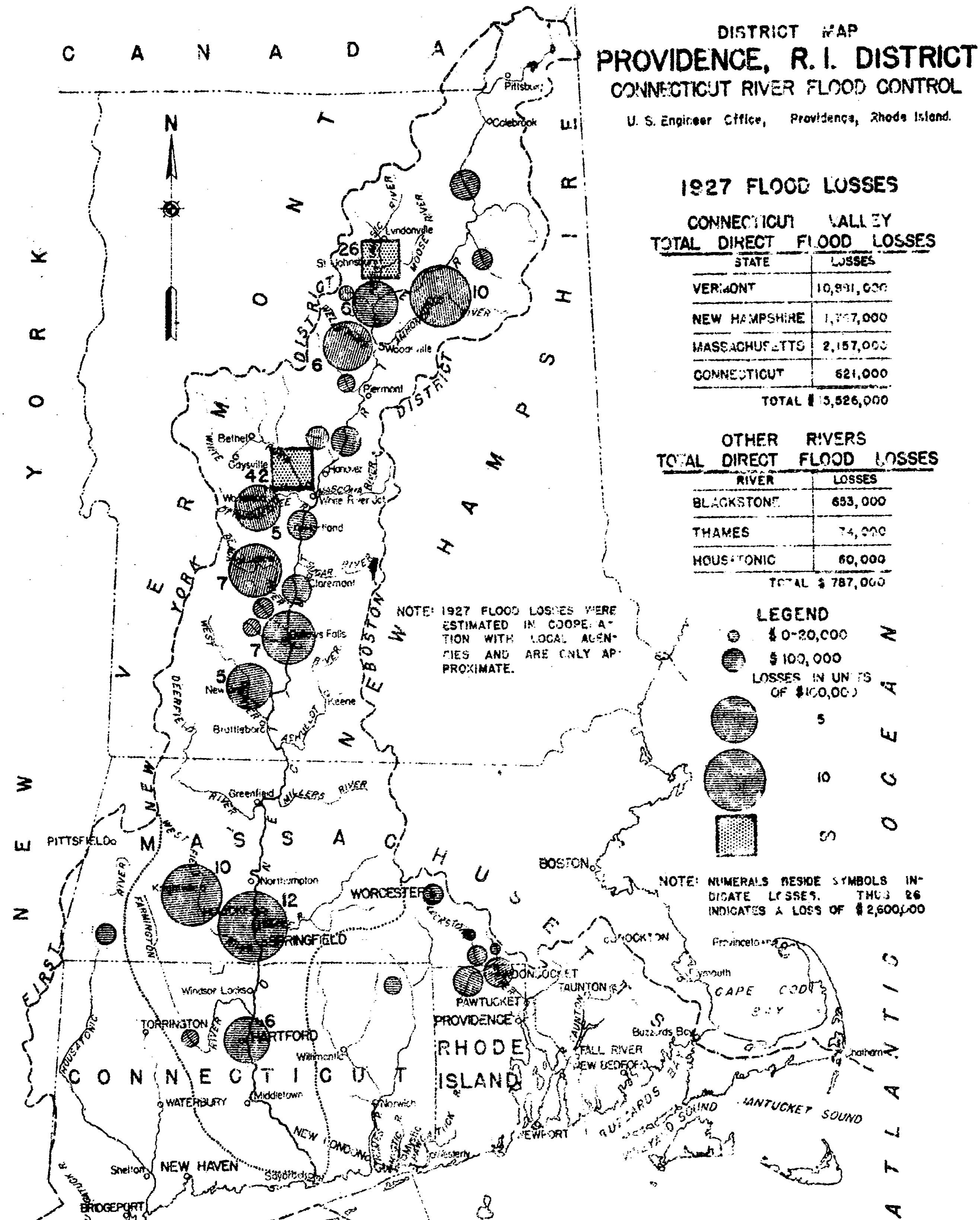
TABLE 30

Estimate of depreciation of property values in flooded towns,
Flood of 1936, Connecticut River Watershed
(Twenty Reservoir Plan)

State	Total Population 1930 Census	Total Pre-Flood Assessed Valuation	Est. Value of Property in Flooded Areas	Est. Depreciation of Property Values
(1)	(2)	(3)	(4)	(5)
Vermont	63,769	$ 69,000,000	$ 12,321,000	$ 1,125,000
New Hampshire	57,065	80,000,000	10,517,000	953,000
Massachusetts	400,146	645,000,000	146,707,000	33,507,000
Connecticut	280,902	529,000,000	161,271,000	39,272,000
Total	801,882	$1,323,000,000	$ 330,816,000	$ 74,857,000

SECTION II

PLATE REFERENCE



DISTRICT MAP

PROVIDENCE, R. I. DISTRICT

CONNECTICUT RIVER FLOOD CONTROL

U. S. Engineer Office, Providence, Rhode Island.

1927 FLOOD LOSSES

CONNECTICUT VALLEY
TOTAL DIRECT FLOOD LOSSES

STATE	LOSSES
VERMONT	10,991,000
NEW HAMPSHIRE	1,[illegible]7,000
MASSACHUSETTS	2,157,000
CONNECTICUT	621,000
TOTAL	$ 15,526,000

OTHER RIVERS
TOTAL DIRECT FLOOD LOSSES

RIVER	LOSSES
BLACKSTONE	653,000
THAMES	[illegible]4,000
HOUSATONIC	60,000
TOTAL	$ 787,000

LEGEND

$ 0-20,000

$ 100,000

LOSSES IN UNITS OF $100,000

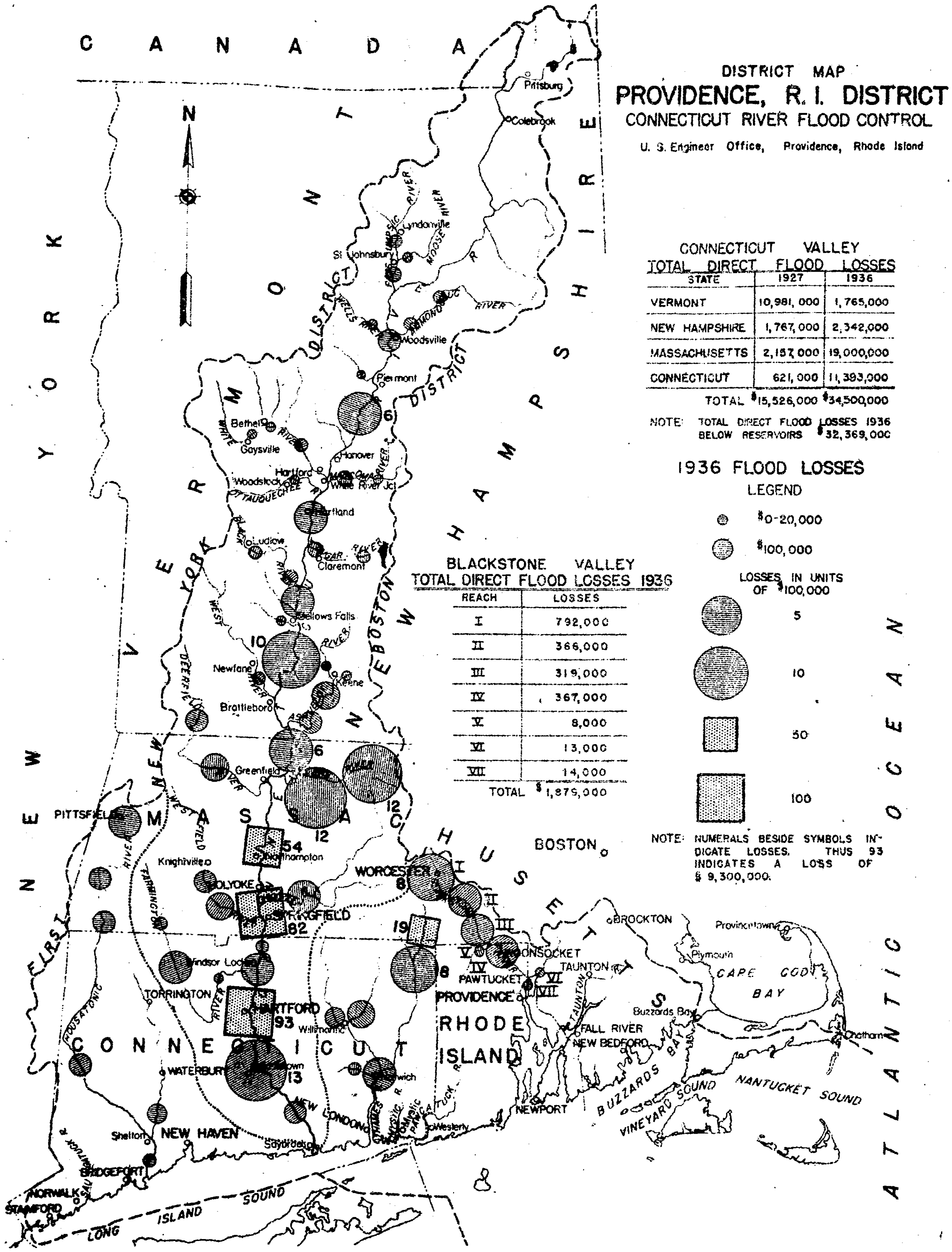
5

10

50

NOTE: 1927 FLOOD LOSSES WERE ESTIMATED IN COOPERATION WITH LOCAL AGENCIES AND ARE ONLY APPROXIMATE.

NOTE: NUMERALS BESIDE SYMBOLS INDICATE LOSSES. THUS 26 INDICATES A LOSS OF $2,600,000



CONNECTICUT VALLEY
TOTAL DIRECT FLOOD LOSSES

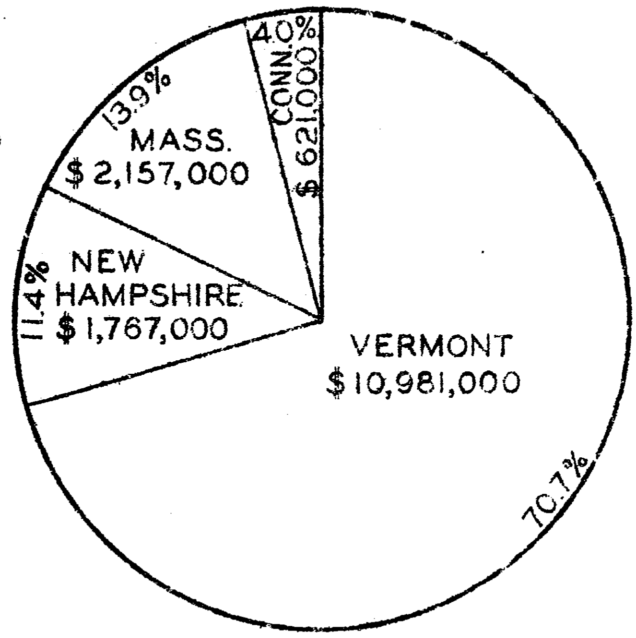
STATE	1927	1936
VERMONT	10,981,000	1,765,000
NEW HAMPSHIRE	1,767,000	2,342,000
MASSACHUSETTS	2,157,000	19,000,000
CONNECTICUT	621,000	11,393,000
TOTAL	$15,526,000	$34,500,000

NOTE: TOTAL DIRECT FLOOD LOSSES 1936 BELOW RESERVOIRS $32,369,000

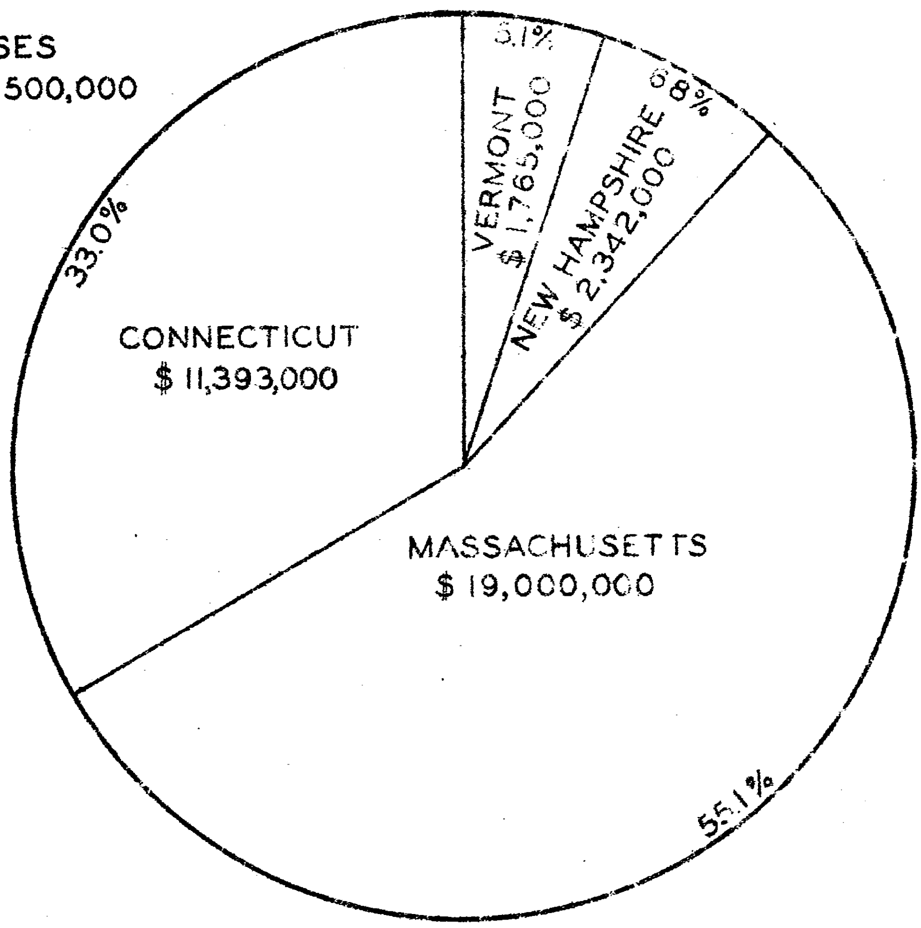
BLACKSTONE VALLEY
TOTAL DIRECT FLOOD LOSSES 1936

REACH	LOSSES
I	792,000
II	366,000
III	319,000
IV	367,000
V	8,000
VI	13,000
VII	14,000
TOTAL	$1,879,000

1927 LOSSES
TOTAL = $ 15,526,000
= 100%



1936 LOSSES
TOTAL = $ 34,500,000
= 100%



CONNECTICUT RIVER FLOOD CONTROL

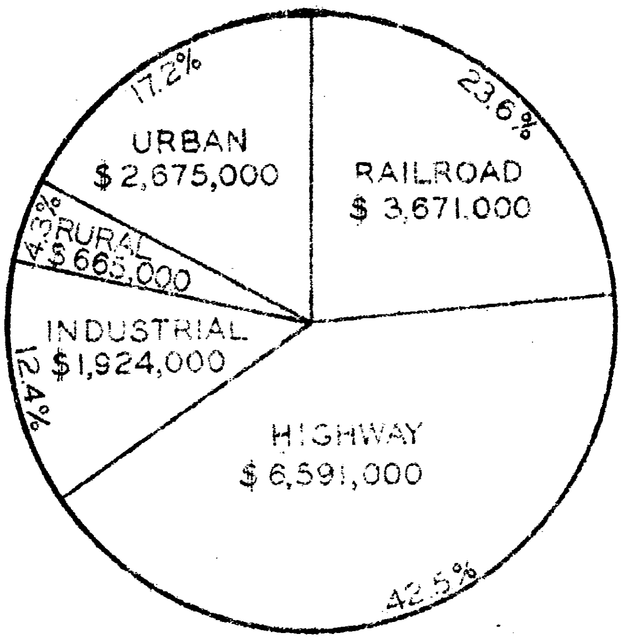
DIRECT FLOOD LOSSES

COMPARISON OF 1927 & 1936 LOSSES BY STATES

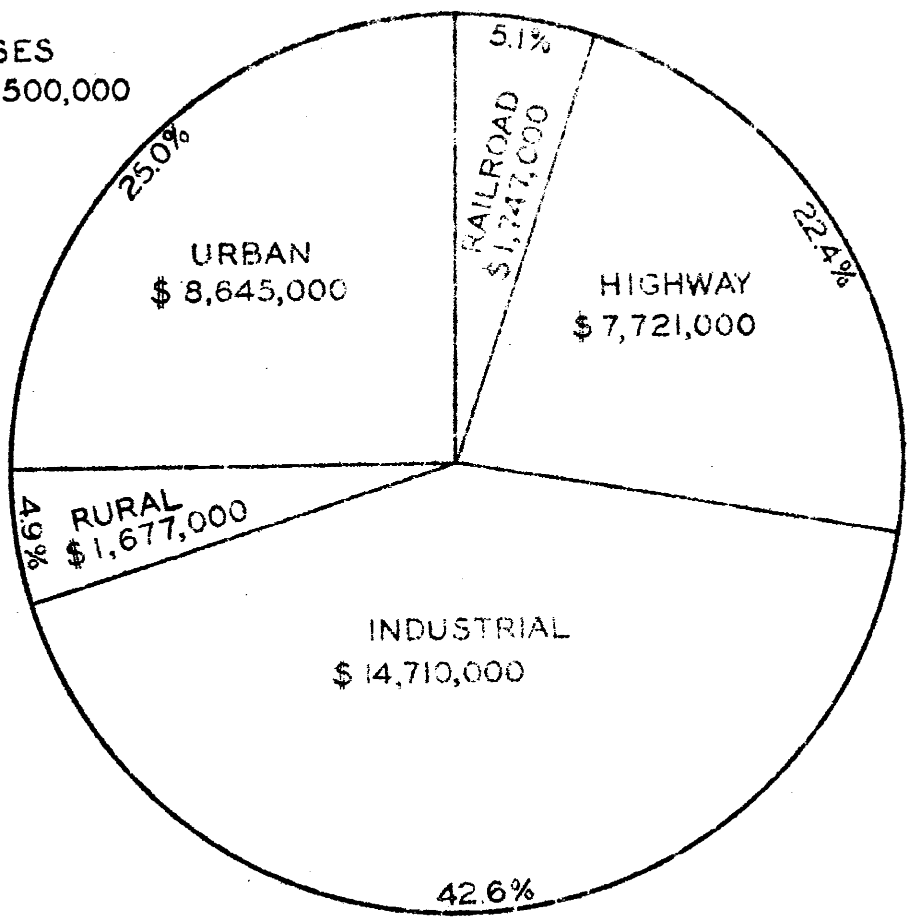
CONNECTICUT RIVER WATERSHED

U. S. ENGINEER OFFICE
PROVIDENCE, R. I.

1927 LOSSES
TOTAL = $ 15,526,000
= 100%



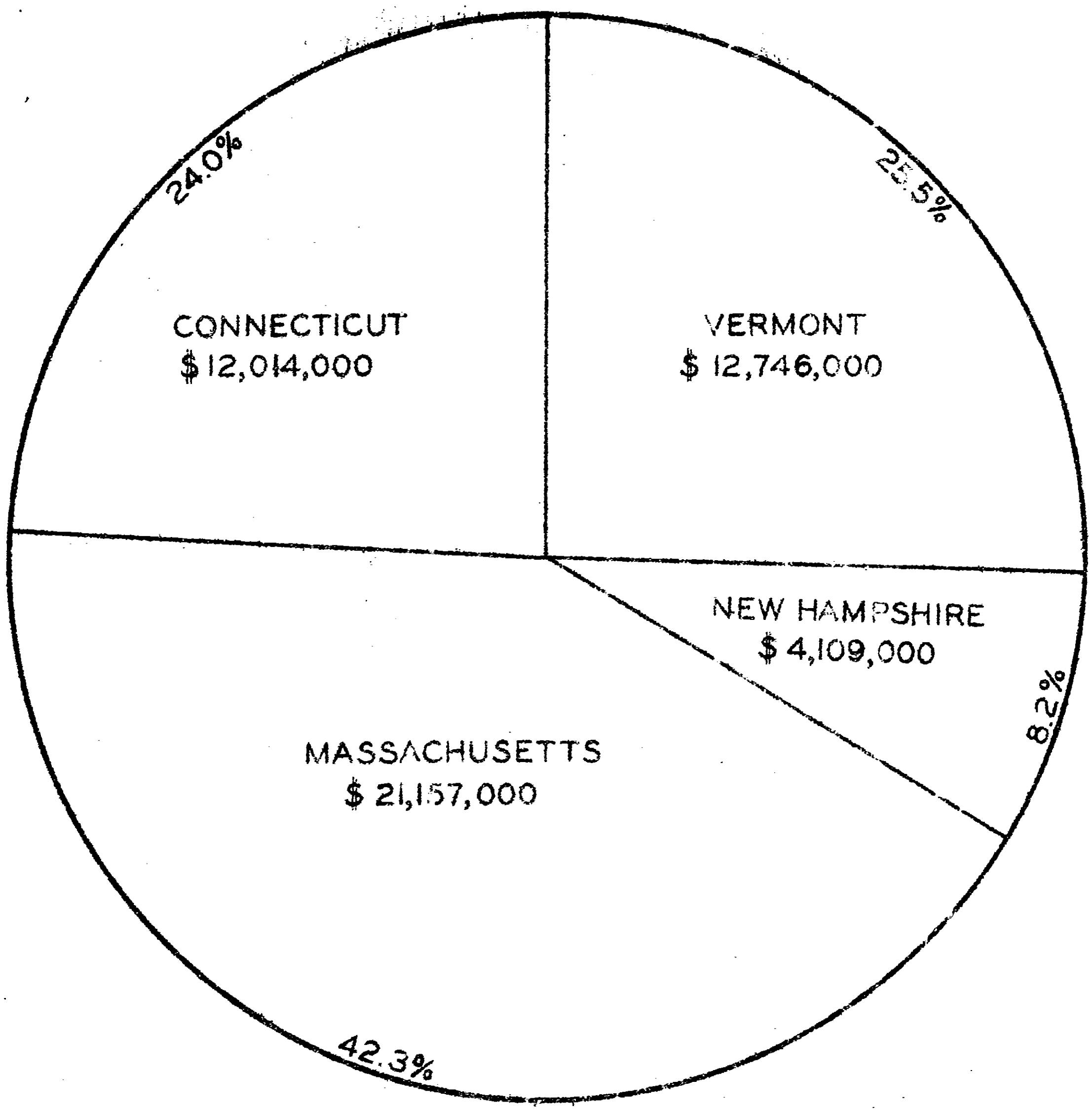
1936 LOSSES
TOTAL = $ 34,500,000
= 100%



CONNECTICUT RIVER FLOOD CONTROL

DIRECT FLOOD LOSSES
COMPARISON OF 1927 & 1936
LOSSES BY TYPE OF LOSS

CONNECTICUT RIVER WATERSHED
U.S. ENGINEER OFFICE
PROVIDENCE, R.I.



TOTAL 1927 AND 1936 LOSSES = $ 50,026,000 = 100%

CONNECTICUT RIVER FLOOD CONTROL

DIRECT FLOOD LOSSES

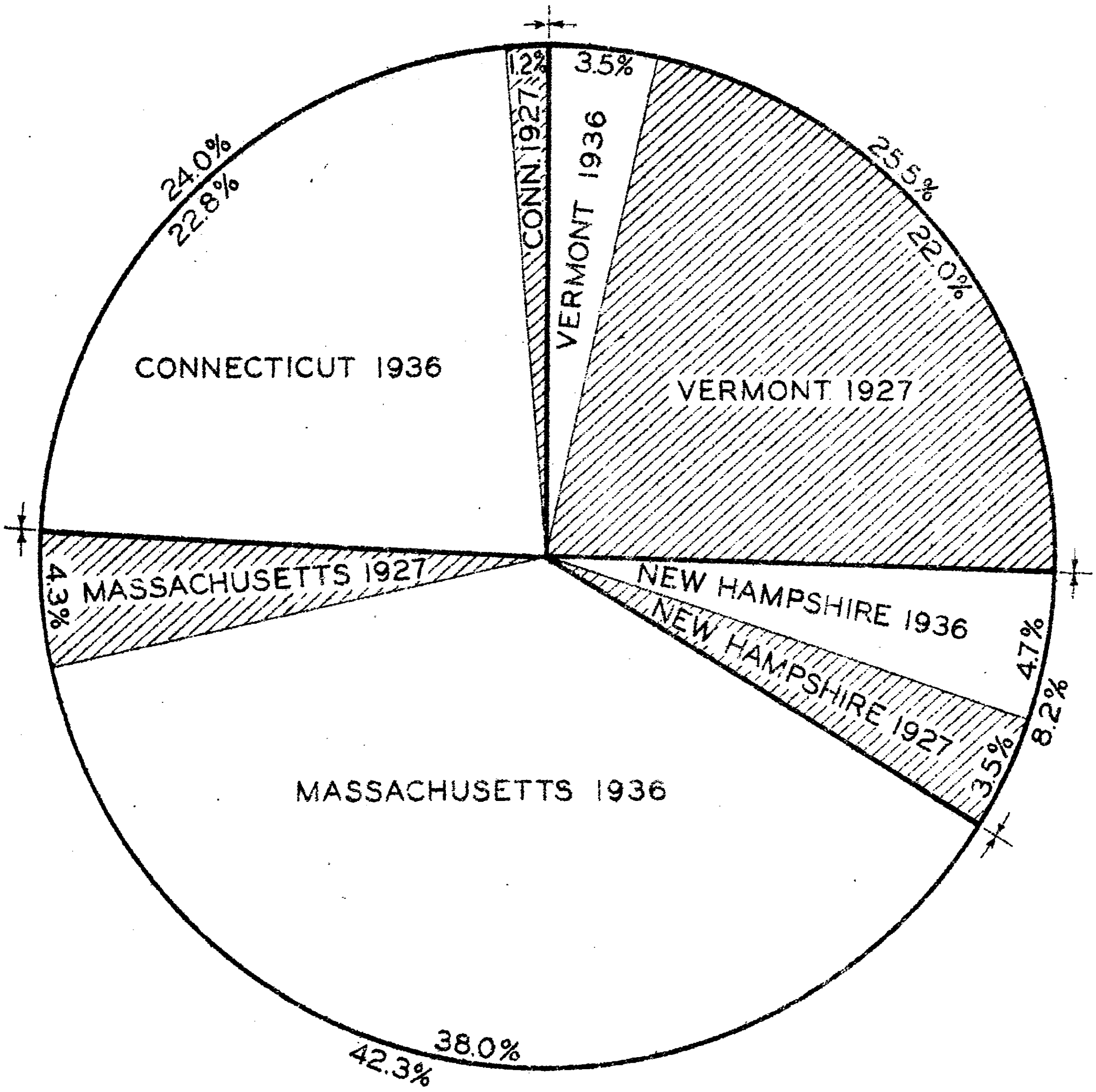
TOTAL 1927 AND 1936

BY STATES

CONNECTICUT RIVER WATERSHED

U. S. ENGINEER OFFICE

PROVIDENCE, R. I.



TOTAL 1927 AND 1936 LOSSES = $ 50,026,000 = 100%

CONNECTICUT RIVER FLOOD CONTROL

DIRECT FLOOD LOSSES

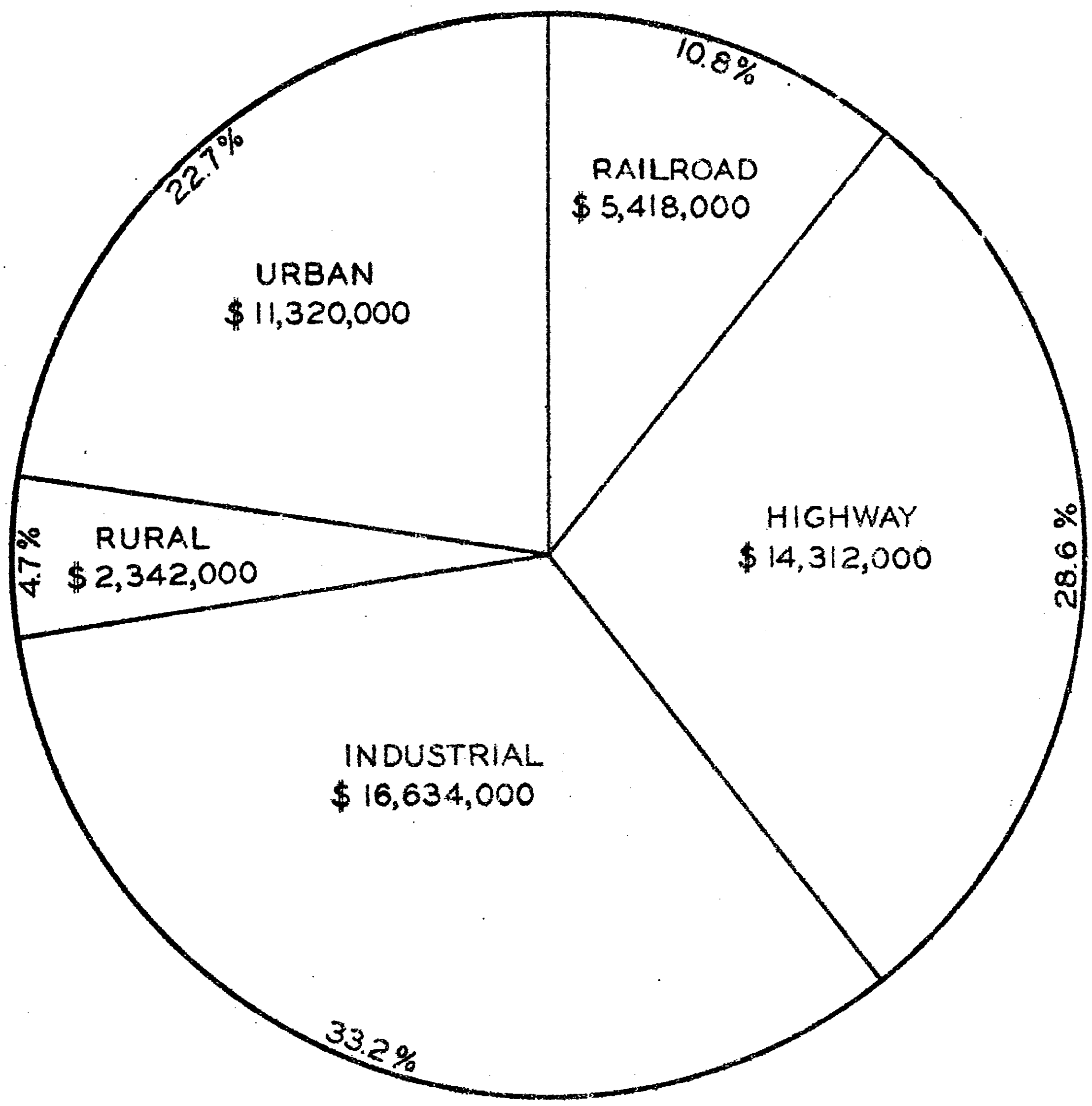
TOTAL 1927 AND 1936

BY STATES

CONNECTICUT RIVER WATERSHED

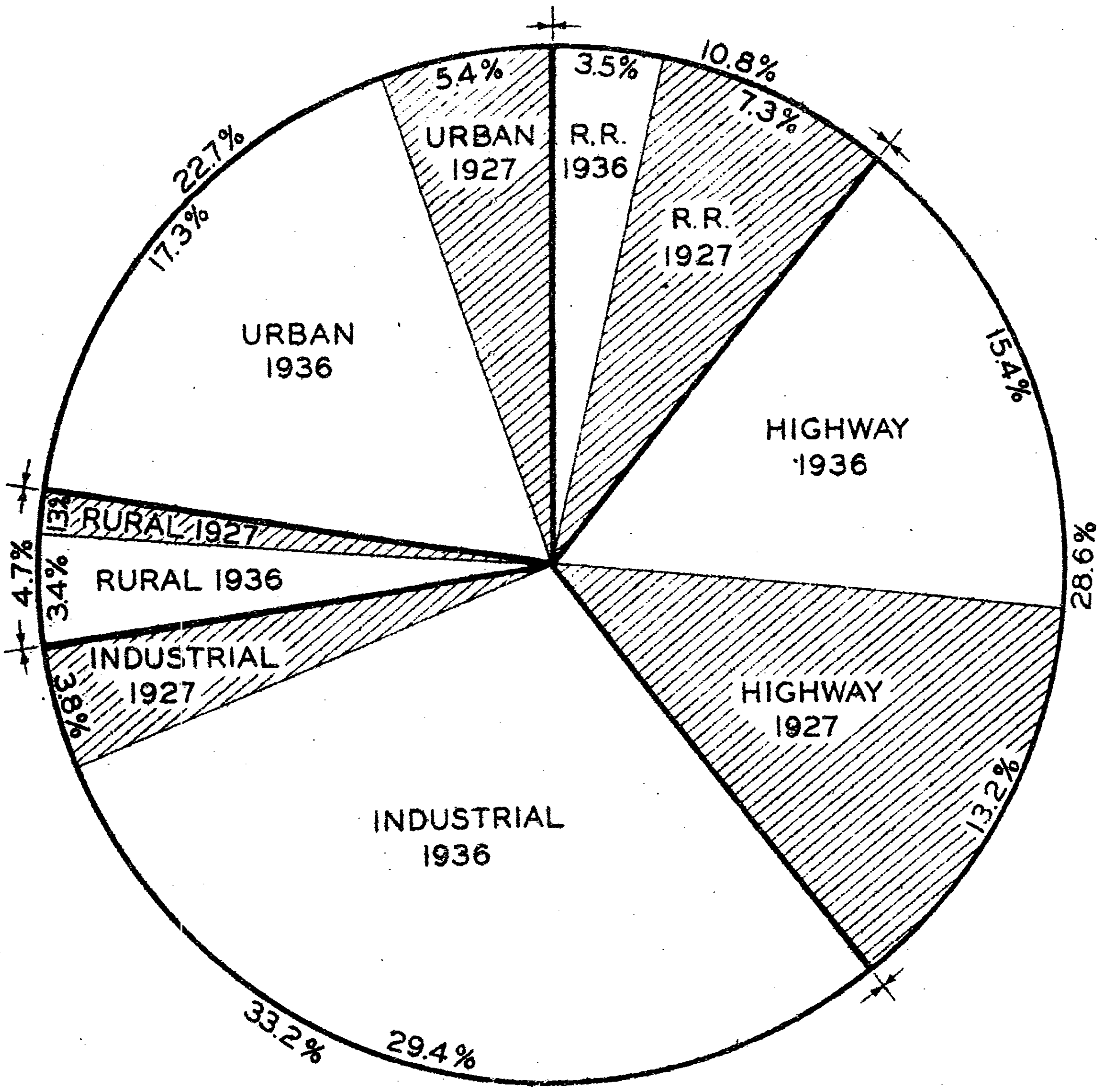
U. S. ENGINEER OFFICE

PROVIDENCE, R. I.



TOTAL 1927 AND 1936 LOSSES = $ 50,026,000 = 100%

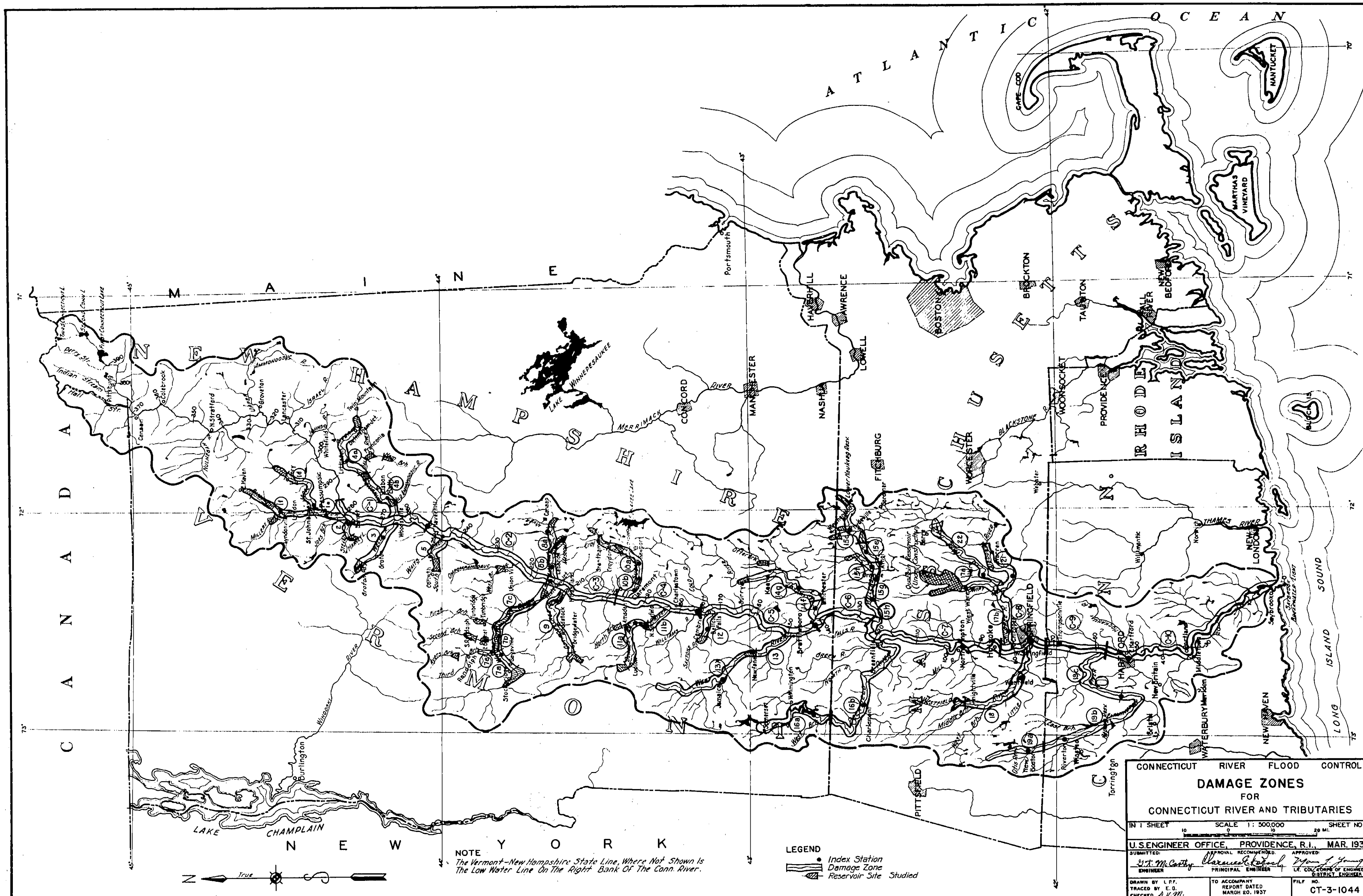
CONNECTICUT RIVER FLOOD CONTROL
DIRECT FLOOD LOSSES
TOTAL 1927 AND 1936 BY
TYPE OF LOSS
CONNECTICUT RIVER WATERSHED
U. S. ENGINEER OFFICE
PROVIDENCE, R. I.

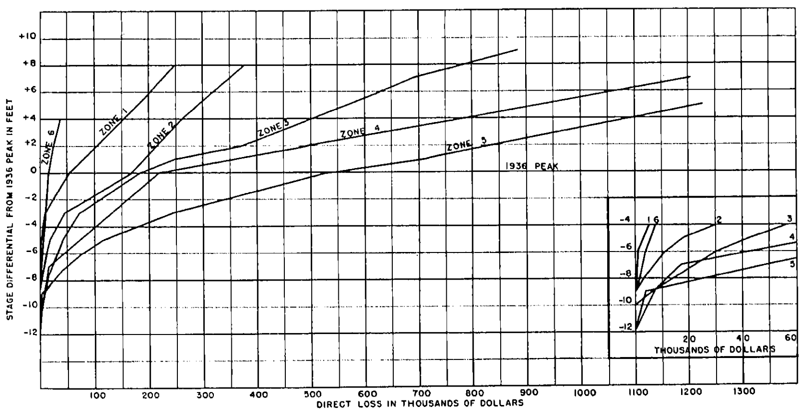


TOTAL 1927 AND 1936 LOSSES = $ 50,026,000 = 100%

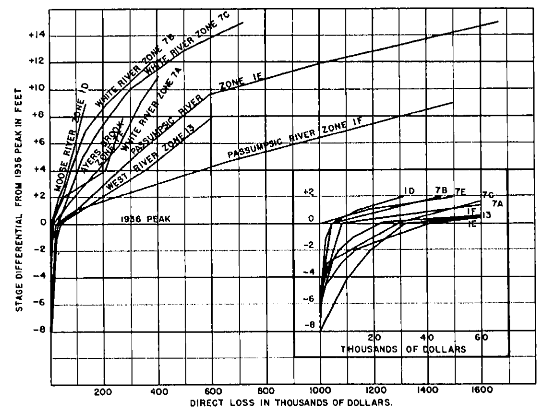
CONNECTICUT RIVER FLOOD CONTROL
DIRECT FLOOD LOSSES
TOTAL 1927 AND 1936 BY
TYPE OF LOSS
CONNECTICUT RIVER WATERSHED
U. S. ENGINEER OFFICE
PROVIDENCE, R. I.

WAR DEPARTMENT CORPS OF ENGINEERS, U.S.ARMY

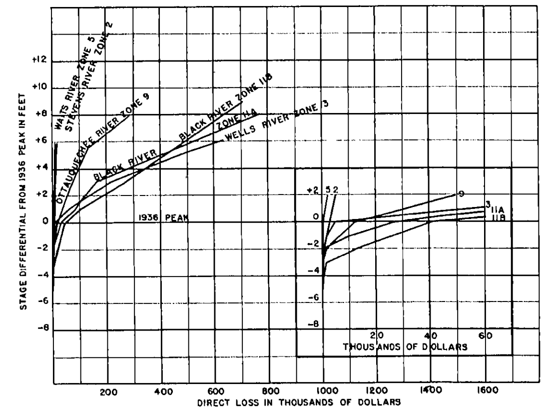




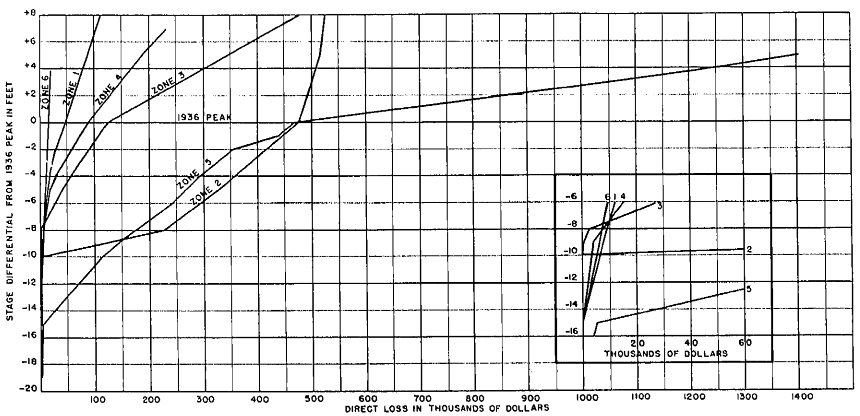
CONNECTICUT RIVER-STATE OF VERMONT



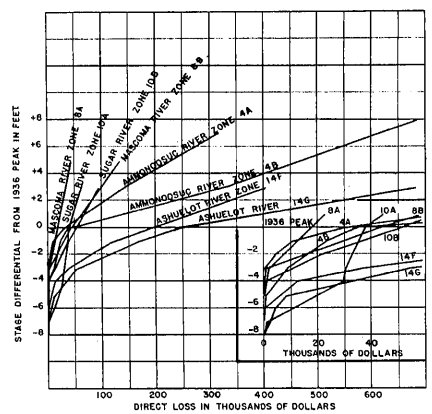
TRIBUTARIES-STATE OF VERMONT



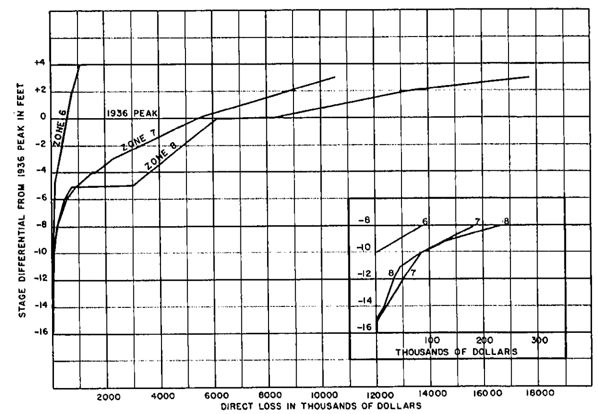
TRIBUTARIES-STATE OF VERMONT



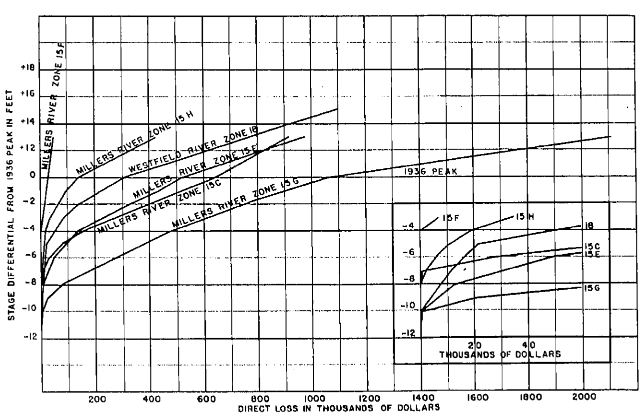
CONNECTICUT RIVER-STATE OF NEW HAMPSHIRE



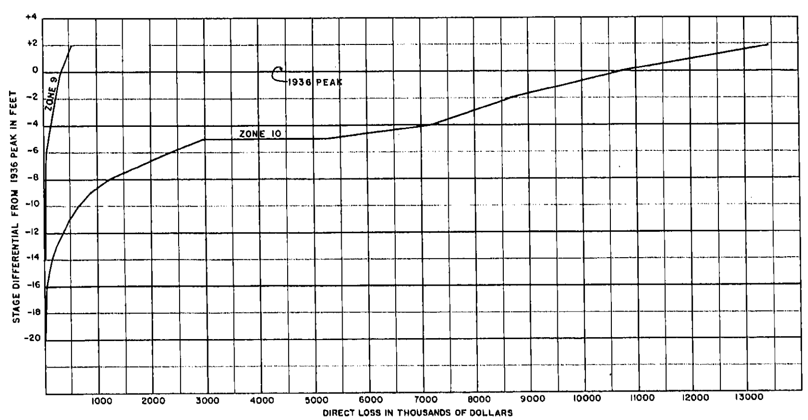
TRIBUTARIES-STATE OF NEW HAMPSHIRE



CONNECTICUT RIVER-STATE OF MASSACHUSETTS



TRIBUTARIES-STATE OF MASSACHUSETTS



CONNECTICUT RIVER-STATE OF CONNECTICUT

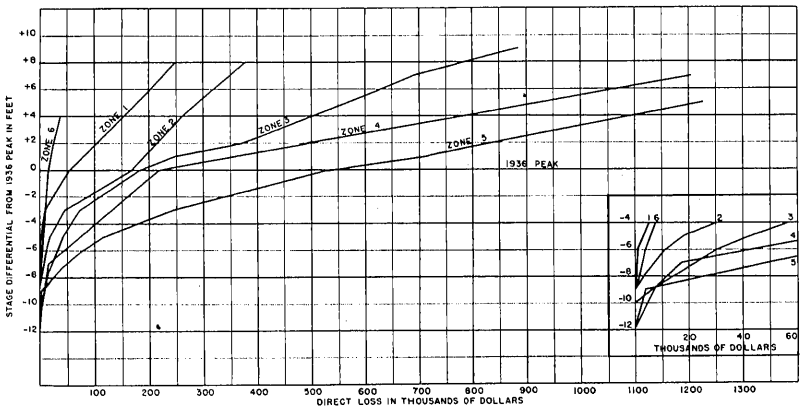
CONNECTICUT RIVER FLOOD CONTROL

STAGE LOSS CURVES

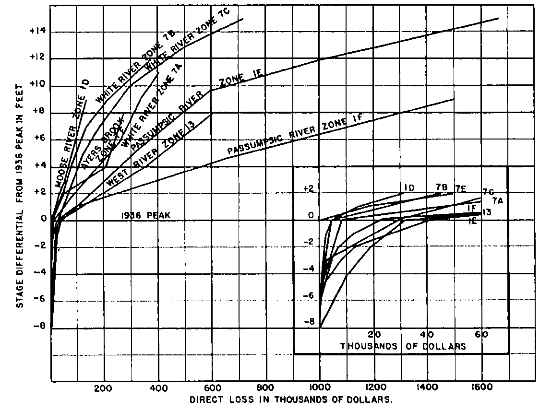
DIRECT, RECURRING LOSSES

1936 FLOOD

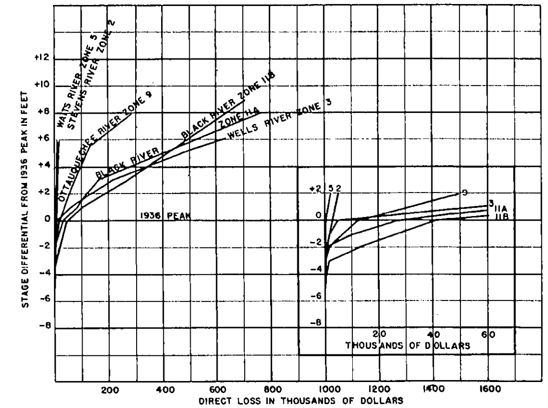
IN 1 SHEET	SCALE AS SHOWN	SHEET NO. 1
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.,		MAR. 1937
SUBMITTED: A. E. ... INSPECTOR	APPROVAL RECOMMENDED: Clarence ... PRINCIPAL ENGINEER	APPROVED: ... LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER
DRAWN BY: A. B. G. TRACED BY: J. F. J. & V. A. T.	TO ACCOMPANY REPORT DATED MARCH 20, 1937	FILE NO. CT-7-1007



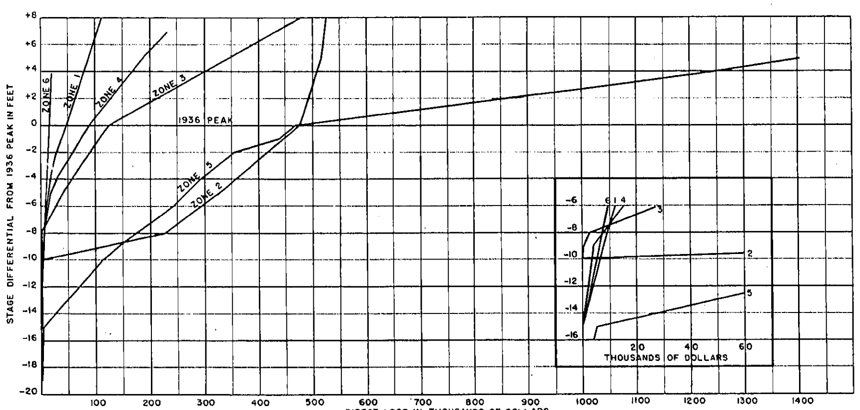
CONNECTICUT RIVER-STATE OF VERMONT



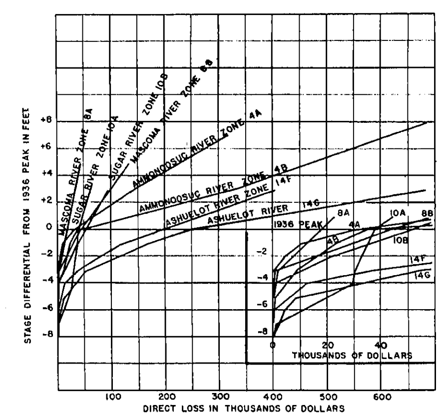
TRIBUTARIES-STATE OF VERMONT



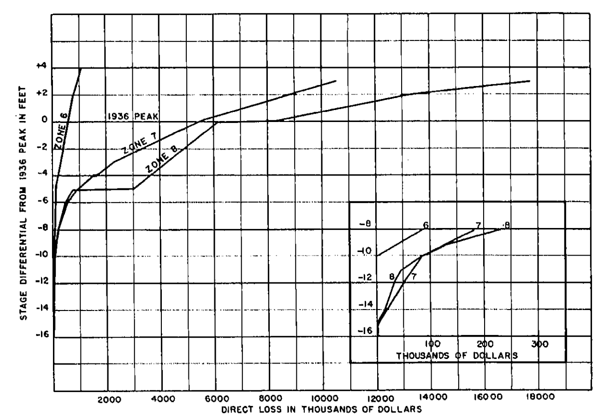
TRIBUTARIES-STATE OF VERMONT



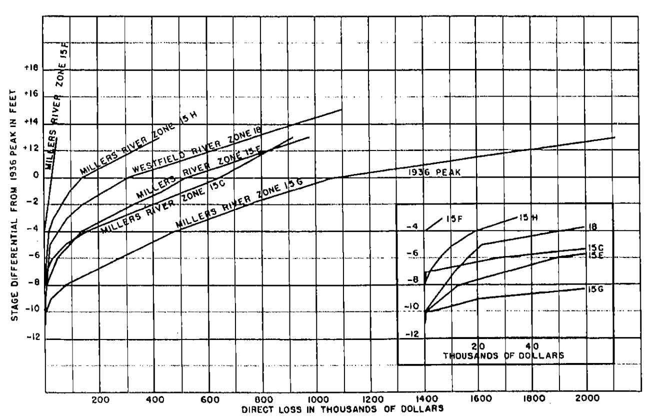
CONNECTICUT RIVER-STATE OF NEW HAMPSHIRE



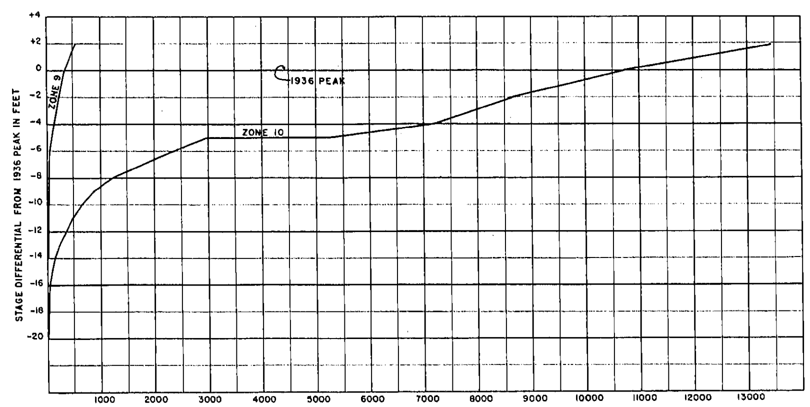
TRIBUTARIES-STATE OF NEW HAMPSHIRE



CONNECTICUT RIVER-STATE OF MASSACHUSETTS



TRIBUTARIES-STATE OF MASSACHUSETTS



CONNECTICUT RIVER-STATE OF CONNECTICUT

CONNECTICUT RIVER FLOOD CONTROL

STAGE LOSS CURVES

DIRECT, RECURRING LOSSES

1936 FLOOD

IN 1 SHEET	SCALE AS SHOWN	SHEET NO. 1
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.		MAR. 1937
SUBMITTED: A. E. Hixon, INSPECTOR	APPROVAL RECOMMENDED: Clarence ..., PRINCIPAL ENGINEER	APPROVED: ..., LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER
DRAWN BY A.B.B. TRACED BY: J.F.J. & V.A.T. CHECKED BY	TO ACCOMPANY REPORT DATED MARCH 20, 1937	FILE NO. CT-7-1007

SECTION III

TABLE REFERENCE

TABLE 34

ANALYSIS OF POTENTIAL POWER DEVELOPMENT AT CONNECTICUT RIVER FLOOD CONTROL DAMS

IDENTIFICATION NO.	RESERVOIR	RIVER	DRAINAGE AREA SQ. MI.	RESERVOIR CAPACITY: Flood Control In.	RESERVOIR CAPACITY: Flood Control A.F.	RESERVOIR CAPACITY: Power (See Note) In.	RESERVOIR CAPACITY: Power (See Note) A.F.	RESERVOIR CAPACITY: Total In.	RESERVOIR CAPACITY: Total A.F.	WATER SURFACE ELEVATION: Min. (Feet Above M.S.L.)	WATER SURFACE ELEVATION: Max. For Power (Feet Above M.S.L.)	WATER SURFACE ELEVATION: Total (Feet Above M.S.L.)	HEADS AVAILABLE FOR POWER: Min. (Feet)	HEADS AVAILABLE FOR POWER: Max. (Feet)	HEADS AVAILABLE FOR POWER: Mean (Feet)	PLANT CAPACITY: Discharge Mean C.F.S.	PLANT CAPACITY: Discharge C.F.S./Sq. Mi.	PLANT CAPACITY: Discharge Total C.F.S.	PLANT CAPACITY: Cap. At Max. Head KW	PRIME OUTPUT: Min. Reg. Discharge C.F.S.	PRIME OUTPUT: Output At Min. Head KW	PRIME OUTPUT: Prime Output Thousand KW.H.	TOTAL OUTPUT AT MEAN HEAD Thousand KW.H.	SECONDARY OUTPUT Thousand KW.H.	ANNUAL TOTAL VALUE Dollars	ANNUAL COST Dollars	RATIO OF VALUE TO COST
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
18A	East Haven	Passumpsic	47.5	6.1	15,500	0	0	6.1	15,500	953	0	1040			0												0
21A	Lyndon Center	Millers Run	52	6.0	16,600	0	0	6.0	16,600	701	0	766.5			0												0
22A	Victory	Moose	66	7.0	24,600	12.5	44,000	17.0	59,800	1120	1159	1166	10	39	24	120	4.0	264	700	100	67	590	1,420	830	7,210	11,400	.63
50	Harvey Lake	Stevens	24.9	5.9	7,800	0	0	5.9	7,800	880	0	900			0												0
24A	Bethlehem Junc.	Ammonoosuc	90	6.0	28,800	0	0	6.0	28,800	1210	0	1356			0												0
26	Gale River	Gale	86	2.9	13,400	0	0	2.9	13,400	839	0	912			0												0
69	Bath	Ammonoosuc	397	6.0	127,000	0	0	6.0	127,000	460	0	600			0												0
27A	Groton Pond	Wells	17.3	7.0	6,500	8.75	8,000	14.0	12,900	1061.5	1087.5	1095.3	10	26	22	30	5.0	85	150	25	17	149	390	241	1,865	4,800	.39
28A	South Branch	S. Branch (Waits)	45	6.0	14,400	6	14,400	10.8	25,900	739	810	829	20	71	50	70	6.0	270	1,300	52	70	620	2,060	1,440	9,280	18,600	.50
48	Union Village	Ompompanoosuc	126	4.5	30,200	0	0	4.5	30,200	417	0	543			0												0
29A	Gaysville	White	226	6.5	77,800	5.5	66,300	10.9	131,200	708	787	821	120	199	190	407	5.7	1,300	17,600	220	1,790	15,700	46,000	30,300	216,500	214,000	1.01
30A	Ayers Brook	Ayers Brook	30	6.0	9,800	5.0	8,000	10.0	16,000	642	695	705	20	53	45	46	5.5	165	600	31	42	370	1,220	850	5 510	10,200	.54
49A	So. Tunbridge	First Branch	102	4.5	24,500	0	0	4.5	24,500	489	0	553			0												0
70	Centerville	White	271	10.8	155,000	0	0	10.8	155,000	370	0	508			0												0
66	West Canaan	Mascoma	80	6.0	25,700	6.25	26,700	11.0	47,000	860	894		10	34	20	124	6.0	480	1,100	94	64	560	1,470	910	7,210	16,200	.45
72	Mascoma Lake	Mascoma	73	4.4	17,000	0	0	4.4	17,000	737	0	750			0												0
63	No. Hartland	Ottauquechee	222	4.1	48,500	0	0	4.1	48,500	394	0	528			0												0
53A	Stocker Pond	Stocker Brook	35	6.0	11,300	5.88	11,000	10.7	20,000	1000	1032	1040	20	32	26	55	5.5	190	410	40	54	470	850	380	4,900	7,920	.62
64A	Claremont	Sugar	245	4.6	60,000	0	0	4.6	60,000	522	0	607			0												0
36	Ludlow	Black	56	4.5	13,400	0	0	4.5	13,400	999	0	1057			0												0
74	Perkinsville	Black	142	6.0	46,200	11.1	84,000	15.6	118,000	520	666	681	60	146	115	213	6.0	852	8,400	146	590	5,200	14,500	9,300	69,500	103,800	.67
55A	No. Springfield	Black	156	3.2	26,500	0	0	3.2	26,500	451	0	519			0												0
40A	Newfane	West	326	6.0	105,000	8.30	144,200	12.8	222,500	368	503	530	50	135	110	585	6.0	1,950	17,900	450	1,520	13,300	38,200	24,900	181,100	217,800	.83
57A	Surry Mountain	Ashuelot	100	6.0	32,000	0	0	6.0	32,000	484	0	541			0												0
59	Lower Naukeag	Millers	19.7	5.1	5,400	0	0	5.1	5,400	1062	0	1076			0												0
60	Hydeville	Millers	65.3	4.2	14,700	0	0	4.2	14,700	830	0	875			0												0
61A	Priest Pond	Priest Pd. Brook	19	6.0	6,000	12.5	12,700	17.0	17,200	850	887	895	10	37	25	33	5.8	110	275	31	21	184	490	306	2,390	6,300	.38
65	Birch Hill	Millers	156.3	6.0	50,000	0	0	6.0	50,000	815	0	847			0												0
62A	Tully	Tully	50	8.0	21,300	12.5	33,300	18.0	48,000	620	678	688	15	66	35	90	6.3	315	1,400	83	84	740	1,870	1,130	9,310	19,800	.47
47	Knightville	Westfield	164	4.5	39,300	1.5	13,160	6.0	52,600	500	554	610	65	124	105	230	3.66	600	5,000	100	440	3,850	14,300	10,450	62,150	63,000	.99

NOTE: Power Storage = (Total Capacity - Flood Control Capacity) + [.25 (Total Capacity - Flood Control Capacity)] *

* Max. Value = 3.0" or (Flood Control Capacity - 4.5")

CT - 9 - 1084

TABLE 35 — BASIC DATA FOR ELECTRIC POWER PLANTS IN CONNECTICUT RIVER WATERSHED — EXISTING AND COMPREHENSIVE DEVELOPMENTS

POWER PLANT	RIVER	Drainage Area	Served by Reservoir	EXISTING DEVELOPMENT									COMPREHENSIVE DEVELOPMENT								
				Net Head	Installed Capacity	Load Factor	Flows Full Load	Flows At Load Factor	Existing Storage		Existing Min. Flow		Net Head	Installed Capacity	Load Factor	Flows Full Load	Flows At Load Factor	Existing at Pittsburg at Upper 15 Mile Falls Storage		Min. Flow	
		Sq. Mi.	Number	Feet	K W	%	c.f.s.	c.f.s.	A.F.	A.F./Sq. Mi.	c.f.s.	c.f.s./Sq. Mi.	Feet	K W	%	c.f.s.	c.f.s.	A.F.	A.F./Sq. Mi.	c.f.s.	c.f.s./Sq. Mi.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Lyndonville	Passumpsic	221		14.5	60	30	56	17	0		44	.20				Same as Existing					
"	"	225		58.5	600	30	143	43	0		45	.20				"	"	"			
Twin	"	250		16.3	250	30	216	65	0		46	.20				"	"	"			
State	"	256		18.2	350	30	270	81	0		51	.20				"	"	"			
Gas	"	376	22A	9.1	150	30	272	70	0		75	.20				"	"	"			
and	"	420	22A	16.1	875	30	760	227	0		84	.20				"	"	"			
Electric	"	424	22A	22.9	700	30	433	130	0		85	.20									
Barnet, Vt.	Stevens	48		82.0	200	30	36	11	0		10	.20									
Bethlehem, N.H.	Ammonoosuc	99		44.0	300	30	97	30	0		20	.20									
Littleton	"	120		14.0	100	30	97	30	0		24	.20									
Lisbon, N.H.	"	287		15.5	300	30	286	86	0		57	.20									
Boltonville, Vt.	Wells	87	27A	63.5	470	30	110	33	0		17	.20									
Bradford, Vt.	Waits	154		70.0	360	30	76	23	0		31	.20									
Gaysville	White	226	29A										190.0	17,600	30	1,300	390			45	.20
C. V. Public Service	"	411	29A, 30A	12.4	560	30	630	188			82	.20	12.4	560	30	630	188			82	.20
Sharon	"	649	29A, 30A										57.0	6,000	30	1,550	470			130	.20
West Hartford	"	683	29A, 30A										35.0	4,000	30	1,700	500			137	.20
Hartford	"	708	29A, 30A										42.0	5,000	30	1,750	520			142	.20
Lebanon, No.1, N.H.	Mascoma	194	66	17.8	150	30	124	37	22,400	115.0	140	.72									
" , No.2, N.H.	"	194	66	15.4	140	30	134	40	22,400	115.0	140	.72									
" , No.4, N.H.	"	194	66	68.0	1,050	30	230	69	22,400	115.0	140	.72									
Claremont	Sugar	250	53A	22.8	250	30	163	49	19,800	79.0	160	.64				Same as Existing					
Cavendish, Vt.	Black	82		115.0	1,500	30	192	58	0		16	.20									
Perkinsville, Vt.	"	115		21.0	368	30	258	78	0		23	.20									
Newfane	West	326	40A										110.0	17,900	30	1,950	585	0		65	.20
Williamsville	"	400	40A										32.0	4,200	30	1,950	580	0		80	.20
West Dummerston	"	408	40A	20.0	620	30	460	138	0		82	.20	52.0	6,800	30	1,950	580	0		82	.20
Brattleboro	"	420	40A										58.5	7,600	30	1,950	580	0		84	.20
Winchendon, No.1, Mass.	Millers			19.5	350	30	265	80	0			.20				Same as Existing					
" , No.3, Mass.	"			16.3	200	30	180	54	0			.20				"	"	"			
Wendell, Mass.	"		61A, 62A	21.1	1,120	30	780	234	0			.20				"	"	"			
Farley, Mass.	"	377	61A, 62A	17.1	360	30	310	93	0		75	.20				"	"	"			
MacIndoes	Connecticut	2233	22A	30.8	10,000	60	4,530	2,720	88,000	39.4	1,060	.48	30.8	10,000	60	4,530	2,720	301,000	135	1,710	.77
East Ryegate	"	2245	22A	12.6	2,000	50	2,320	1,160	88,000	39.1	1,060	.47	12.6	2,000	50	2,320	1,160	301,000	134	1,710	.76
Piermont	"	3104	22A, 27A										27.5	13,000	40	7,000	2,800	301,000	95	1,880	.61
Wilder	"	3367	22A, 27A	35.0	3,120	40	1,240	496	88,000	25.7	1,280	.38	35.0	17,500	40	7,600	3,040	301,000	89	1,930	.57
Hart Island		4573	Above & 29A, 30A, 66										26.0	18,500	40	10,500	4,200	301,000	66	2,170	.47
Bellows Falls	"	5387	Above & 53A & 74	60.0	45,000	50	10,500	5,250	88,000	16.1	1,690	.31	60.0	45,000	50	10,500	5,250	301,000	56	2,340	.43
Vernon	"	6239	Above & 40A	34.2	28,000	40	11,400	4,570	88,000	14.0	1,860	.30	34.2	28,000	40	11,400	4,570	301,000	48	2,510	.40
Turners Falls	"	7138	Above & 61A, 62A	64.0	57,000	40	12,600	5,050	88,000	12.2	2,040	.30	64.0	57,000	40	12,600	5,050	301,000	42	2,690	.38
*Holyoke, No.1 and 2	"	8242	Above	22.8	11,000	40	7,750	3,100	266,000	32.0	2,860	.35									
Enfield	"	9655	Above	28.0	2,900	40	1,150	508	266,000	27.3	3,140	.33	28.0	35,000	40	18,500	7,400	479,000	50	3,790	.39

*Large volume of water sold to industries.

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TABLE 36

POWER VALUE TO DOWNSTREAM PLANTS OF ONE INCH OF CONSERVATION STORAGE AT FLOOD CONTROL RESERVOIRS

						EXISTING DEVELOPMENTS										COMPREHENSIVE DEVELOPMENTS									
IDENTIFICATION NO.	RESERVOIR	RIVER	DRAINAGE AREA (SQ. MI.)	FLOOD CONTROL CAPACITY (INCHES)	POWER STORAGE CAPACITY (INCHES)	ACCUM. USABLE HEAD (FEET)	INCREASED YEARLY OUTPUT (THOUS. KWH.)	VALUE OF INCREASED YEARLY OUTPUT ($)	INCREASE OF MIN. LOW WATER FLOW (C.F.S.)	ACCUM. CAP. SERVED BY ONE C.F.S. (KW/CFS)	INCREASED PEAKING CAPACITY (KW)	VALUE OF INCREASED PEAK. CAP. ($)	ANNUAL COST OF STORAGE ($)	RATIO OF ENERGY BENEFIT TO COST	RATIO OF ENERGY & POWER BENEFIT TO COST	ACCUM. USABLE HEAD (FEET)	INCREASED YEARLY OUTPUT (THOUS. KWH.)	VALUE OF INCREASED YEARLY OUTPUT ($)	INCREASE OF MIN. LOW WATER FLOW (C.F.S.)	ACCUM. CAP. SERVED BY ONE C.F.S. (KW/CFS)	INCREASED PEAKING CAPACITY (KW)	VALUE OF INCREASED PEAK. CAP. ($)	ANNUAL COST OF STORAGE ($)	RATIO OF ENERGY BENEFIT TO COST	RATIO OF ENERGY & POWER BENEFIT TO COST
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
18A	East Haven	Passumpsic	47.5	6.1	0																				
21A	Lyndon Center	Millers Run	52	6.0	0																				
22A	Victory	Moose	66	7.0	12.50	263.4	699	2,097	11.7	37.35	451	2,706	1,850	1.13	2.59	421.0	1,202	3,606	11.7	69.83	817	4,902	1,850	1.94	4.58
50	Harvey Lake	Stevens	24.9	5.9	0																				
24A	Bethlehem Jct.	Ammonoosuc	90	6.0	0																				
26	Gale River	Gale River	86	2.9	0																				
69	Bath	Ammonoosuc	397	6.0	0																				
27A	Groton Pond	Wells	17.3	7.0	8.75	244.5	183	549	3.0	39.00	117	702	1,144	0.48	1.09	724.3	542	1,626	3.0	119.33	358	2,148	1,144	1.42	3.30
28A	South Branch	So. Branch (Waits)	45	6.0	6.00	158.2	310	930	8.0	26.75	214	1,284	5,830	0.16	0.38	402.9	792	2,376	8.0	68.38	547	3,282	5,830	0.41	0.97
48	Union Village	Ompompanoosuc	126	4.5	0																				
29A	Gaysville	White	226	6.5	5.50	170.4	1,682	5,046	40.0	28.88	1,155	6,930	13,200	0.38	0.91	413.2	4,084	12252	40.0	68.70	2,748	16,488	13200	0.93	2.18
30A	Ayers Brook	Ayers Brook	30	6.0	5.00	170.4	223	669	5.4	27.41	148	888	3,600	0.19	0.43	413.2	540	1,620	5.4	68.52	370	2,220	3,600	0.45	1.07
49A	So. Tunbridge	First Branch	102	4.5	0																				
70	Centerville	White	271	10.8	0																				
66	W. Canaan	Mascoma	80	6.0	6.25	259.4	906	2,718	14.4	43.75	630	3,780	2,160	1.26	3.00	715.5	2,496	7,488	14.4	121.11	1,744	10,464	2,160	347	8.32
72	Mascoma Lake	Mascoma	73	4.4	0																				
63	N. Hartland	Ottauquechee	222	4.1	0																				
53A	Stocker Pond	Stocker Brook	35	6.0	5.88	299.2	459	1,377	6.1	56.72	346	2,076	1,700	0.81	2.03	382.0	586	1,758	6.1	70.82	432	2,592	1,700	1.03	2.56
64A	Claremont	Sugar	245	4.6	0																				
36	Ludlow	Black	56	4.5	0																				
74	Perkinsville	Black	142	6.0	11.10	158.2	978	2,934	25.4	26.70	678	4,068	16,500	0.18	0.42	412.8	2,560	7,680	25.4	69.85	1,775	10,650	16,500	0.47	1.11
55A	N. Springfield	Black	156	3.2	0																				
40A	Newfane	West	326	6.0	8.30	98.2	1,396	4,188	57.8	16.63	961	5,766	15,400	0.27	0.64	323.5	4,600	13,800	57.8	54.81	3,168	19,008	15,400	0.89	2.12
57A	Surry Mtn.	Ashuelot	100	6.0	0																				
59	Lower Naukeag	Millers	19.7	5.1	0																				
60	Hydeville	Millers	65.3	4.2	0																				
61A	Priest Pond	Priest Pond Brook	19	6.0	12.50	102.2	85	255	3.4	17.35	59	354	1,440	0.18	0.42	626.3	522	1,566	3.4	106.18	361	2,166	1,440	1.08	2.59
65	Birch Hill	Millers	156.3	6.0	0																				
62A	Tully	Tully	50	8.0	12.50	85.1	186	558	9.0	14.33	129	774	2,720	0.20	0.49	393.3	859	2,577	9.0	66.11	595	3,570	2,720	0.95	2.26
47	Knightville	Westfield	164	4.5	1.50	0	–	–	29.3	–	–	–	–	–	–	413.9	2,933	8,799	29.3	70.10	2,053	12,318	30,670	0.29	0.69

NOTE: Where no values are shown the physical conditions limit the reservoir capacity to that required for flood control.

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TABLE 36A. RATIOS OF BENEFITS TO COST FROM CONSERVATION STORAGE AT FLOOD CONTROL DAMS

Reservoir	Net Drainage Area Sq.Mi.	Reservoir Capacities: Flood Control (Inches)	Total (Inches)	Conservation (Inches)	Usable Cons. (Inches)	Annual Cost: Flood Control (Thousand Dollars)	Total (Thousand Dollars)	Conservation (Thousand Dollars)	Existing Developments: Annual Value to Power (Thousand Dollars)	Ratio Benefits to Cost: Energy Available	Ratio Benefits to Cost: Energy & Peaking	Cost Mills per K.W.H.	Comprehensive Developments: Annual Value to Power Thous.$	Ratio of Benefits to Cost: Energy Available	Ratio of Benefits to Cost: Energy & Peaking	Cost Mills per K.W.H.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Victory	66	7.0	17.0	10.0	12.5	37.8	61.0	23.2	60.0	1.13	2.59	2.7	106.4	1.94	4.58	1.5
Groton Pond	17.3	7.0	14.0	7.0	8.75	10.2	20.2	10.0	10.9	.48	1.09	6.2	33.0	1.42	3.30	2.1
Gaysville	226	6.5	10.9	4.4	5.5	208.4	281.0	72.6	65.9	.38	.91	7.8	158.1	.93	2.18	3.2
Ayers Brook	30	6.0	10.0	4.0	5.0	43.4	61.4	18.0	7.8	.19	.43	16.1	19.2	.45	1.07	6.7
West Canaan	80	6.0	11.0	5.0	6.25	104.8	119.5	14.6	40.6	1.16	2.78	2.6	112.2	3.21	7.70	.94
Stocker Pond	35	6.0	10.7	4.7	5.88	30.5	40.5	10.0	20.3	.81	2.03	3.7	25.6	1.03	2.56	2.9
Perkinsville	142	6.0	15.6	9.6	11.1	192.0	375.0	183.0	77.7	.18	.42	16.8	203.4	.47	1.11	6.4
Newfane	326	6.0	12.8	6.8	8.3	250.9	379.0	128.1	82.6	.27	.64	11.1	272.3	.89	2.12	3.4
Priest Pond	19	6.0	17.0	11.0	12.5	27.0	45.0	18.0	7.6	.18	.42	16.9	46.6	1.08	2.59	2.8
Tully	50	8.0	18.0	10.0	12.5	36.0	70.0	34.0	16.6	.20	.49	14.6	76.8	.95	2.26	3.2

TABLE 37

ANALYSIS OF POWER BENEFITS AVAILABLE FROM VICTORY STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

Reservoir: 22A
Drainage Area: 66 square miles
Reservoir Capacity Total: 60,000 acre-feet - 17 inches
Reservoir Capacity for Power: 44,000 " " - 12.5 "
Increased Minimum Low-Water Flow: 146 c.f.s.

Name of Plant	Existing Plants: Net Head	Existing Plants: Increased Min. Flow Usable	Existing Plants: Increased Peaking Capacity	Existing Plants: Increased Elec. Energy Thousand	Existing Plants: Avg. Annual Value of Increased Power: Peaking Capacity	Existing Plants: Avg. Annual Value: Electric Energy	Existing Plants: Avg. Annual Value: Total Capacity & Energy	Comprehensive Plan: Net Head	Comprehensive Plan: Increased Min. Flow Available	Comprehensive Plan: Increased Peaking Capacity	Comprehensive Plan: Increased Elec. Energy Thousand	Comprehensive Plan: Avg. Annual Value of Increased Power: Peaking Capacity	Comprehensive Plan: Avg. Annual Value: Electric Energy	Comprehensive Plan: TOTAL Capacity & Energy
	ft.	cfs	k.w.	kwh	dollars	dollars	dollars	ft.	cfs	k.w.	kwh	dollars	dollars	dollars
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Existing Plants														
Twin State Gas)	(9.1)	0	0	0	-	-	-	9.1	146	300	326	$ 1,800	$978	$2,778
& Electric Co.)	16.1	143	525	568	$3,150	$1,704	$4,854	16.1	146	533	580	3,198	1,740	4,938
)	22.9	45	235	254	1,410	762	2,172	22.9	146	760	825	4,560	2,475	7,035
McIndoes	30.8	146	510	1,100	3,060	3,300	6,360	30.8	146	510	1,100	3,060	3,300	6,360
East Ryegate	12.6	100	170	310	1,020	930	1,950	12.6	146	248	450	1,488	1,350	2,838
Bellows Falls	60.0	146	1,190	2,160	7,140	6,480	13,620	60.0	146	1,190	2,160	7,140	6,480	13,620
Vernon	34.2	146	845	1,225	5,070	3,675	8,745	34.2	146	845	1,125	5,070	3,375	8,445
Turners Falls	64	146	1,600	2,300	9,600	6,900	16,500	64	146	1,600	2,300	9,600	6,900	16,500
Holyoke	22.8	146	565	820	3,390	2,460	5,850	22.8	146	565	820	3,390	2,460	5,850
Totals	263.4		5,640	8,737	$33,840	$26,211	$60,051	272.5		6,551	9,686	$39,306	$29,058	$68,364
Redeveloped Plants														
Wilder								35	146	865	1,260	5,190	3,780	8,970
Holyoke								32	146	790	1,150	4,740	3,450	8,190
Enfield								28	146	695	1,005	4,170	3,015	7,185
Future Plants														
Piermont								27.5	146	680	995	4,080	2,985	7,065
Hart Island								26	146	630	935	3,780	2,805	6,585
Total for Redeveloped and Future Plants								148.5		3,660	5,345	21,960	16,035	37,995
TOTALS								421.0		10,211	15,031	$61,266	$45,093	$106,359

TABLE 38

ANALYSIS OF POWER BENEFITS AVAILABLE FROM GROTON POND STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

RESERVOIR NO.:	27A
DRAINAGE AREA:	17 SQUARE MILES
RESERVOIR CAPACITY TOTAL:	12,700 ACRE-FEET - 14 INCHES
RESERVOIR CAPACITY FOR POWER:	8,000 " " - 8.75 "
INCREASED MINIMUM LOW-WATER FLOW:	26 C.F.S.

NAME OF PLANT	EXISTING PLANTS							COMPREHENSIVE PLAN						
		INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER				INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER		
	NET HEAD	MIN. FLOW USABLE	PEAKING CAPACITY	ELEC. ENERGY THOUSAND KWH	PEAKING CAPACITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY	NET HEAD	MIN. FLOW AVAILABLE	PEAKING CAPACITY	ELEC. ENERGY THOUSAND KWH	PEAKING CAPACITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY
	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(6)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
EXISTING PLANTS														
GREEN MT. POWER CO.(RICKERS)	(13.3)	0	-	-	-	-	-	13.3	26	58	87	$348	$261	$609
DO. (BOLTONVILLE)	63.5	26	279	415	$1,674	$1,245	$ 2,919	63.5	26	279	415	1,674	1,245	2,919
ADAMS PAPER CO.	(25)	0	-	-	-	-	-	25.0	26	110	164	660	492	1,152
BELLOWS FALLS	60	26	212	394	1,272	1,182	2,454	60	26	212	394	1,272	1,182	2,454
VERNON	34.2	26	150	224	900	672	1,572	34.2	26	150	224	900	672	1,572
TURNERS FALLS	64	26	280	420	1,680	1,260	2,940	64	26	280	420	1,680	1,260	2,940
HOLYOKE	22.8	26	100	150	600	450	1,050	22.8	26	100	150	600	450	1,050
TOTALS	244.5		1,021	1,603	$6,126	$4,809	$10,935	282.8		1,189	1,854	7,134	5,562	12,696
REDEVELOPED PLANTS														
WILDER								35	26	155	230	930	690	1,620
HOLYOKE								32	26	140	210	840	630	1,470
ENFIELD								28	26	125	183	750	549	1,299
FUTURE PLANTS														
RICKERS POND								127	26	557	830	3,342	2,490	5,832
BOLTONVILLE								65	26	286	425	1,716	1,275	2,991
MILE 2.2								82	26	360	537	2,160	1,611	3,771
ADAMS PAPER CO.								19	26	84	124	504	372	876
PIERMONT								27.5	26	121	180	726	540	1,266
HART ISLAND								26	26	115	170	690	510	1,200
TOTALS FOR REDEVELOPED AND FUTURE PLANTS								441.5		1,943	2,889	11,658	8,667	20,325
TOTALS								724.3		3,132	4,743	$18,792	$14,229	$33,021

TABLE 39

ANALYSIS OF POWER BENEFITS AVAILABLE FROM GAYSVILLE STORAGE
RESERVOIRS TO DOWNSTREAM PLANTS

RESERVOIR:	29A
DRAINAGE AREA:	226 SQUARE MILES
RESERVOIR CAPACITY TOTAL:	131,200 ACRE-FEET - 10.9 INCHES
RESERVOIR CAPACITY FOR POWER:	66,300 " " - 5.5 "
INCREASED MINIMUM LOW-WATER FLOW:	220 C.F.S.

NAME OF PLANTS	EXISTING PLANTS							COMPREHENSIVE PLAN						
		INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER				INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER		
	NET HEAD	MIN. FLOW US-ABLE	PEAKING CA-PACITY	ELEC. ENERGY THOUS-AND	PEAKING CAPAC-ITY	ELECTRIC ENERGY	TOTAL CAPACITY &ENERGY	NET HEAD	MIN. FLOW AVAIL-ABLE	PEAK-ING CA-PACITY	ELEC. ENERGY THOUS-AND	PEAKING CAPAC-ITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY
	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
EXISTING PLANTS														
C. V. PUB. SERVICE	12.2	106	300	320	$1,800	$ 960	$ 2,760	12.2	220	620	670	$3,720	$2,010	$5,730
BELLOWS FALLS	60	220	1,790	3,260	10,740	9,780	20,520	60	"	1,790	3,260	10,740	9,780	20,520
VERNON	34.2	"	1,275	1,860	7,650	5,580	13,230	34.2	"	1,275	1,860	7,650	5,580	13,230
TURNERS FALLS	64	"	2,388	3,480	14,328	10,440	24,768	64	"	2,388	3,480	14,328	10,440	24,768
HOLYOKE	(22.8)	60	597	330	3,582	990	4,572	22.8	"	850	1,240	5,100	3,720	8,820
TOTALS	170.4		6,350	9,250	$38,100	$27,750	$65,850	193.2	"	6,923	10,510	$41,538	$31,530	$73,068
REDEVELOPED PLANTS														
HOLYOKE								32	"	1,190	1,740	7,140	5,220	12,360
ENFIELD								28	"	1,040	1,520	6,240	4,560	10,800
FUTURE PLANTS														
SHARON								57	"	2,130	3,100	12,780	9,300	22,080
WEST HARTFORD								35	"	1,300	1,900	7,800	5,700	13,500
HARTFORD								42	"	1,560	2,280	9,360	6,840	16,200
HART ISLAND								26	"	970	1,410	5,820	4,230	10,050
TOTALS FOR REDEVELOPED AND FUTURE PLANTS								220.0		8,190	11,950	$49,140	$35,850	$84,990
TOTALS								413.2		15,113	22,460	$90,678	$67,380	$158,058

TABLE 40

ANALYSIS OF POWER BENEFITS AVAILABLE FROM AYERS BROOK STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

RESERVOIR: 30A
DRAINAGE AREA: 30 SQUARE MILES
RESERVOIR CAPACITY TOTAL: 16,000 ACRE-FEET - 10 INCHES
RESERVOIR CAPACITY FOR POWER: 8,000 " " - 5 "
INCREASED MINIMUM LOW-WATER FLOW: 27 C.F.S.

NAME OF PLANT	EXISTING PLANTS							COMPREHENSIVE PLAN						
		INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER				INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER		
	NET HEAD	MIN. FLOW USABLE	PEAKING CAPACITY	ELEC. ENERGY THOUS. AND	PEAKING CAPACITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY	NET HEAD	MIN. FLOW AVAILABLE	PEAKING CAPACITY	ELEC. ENERGY THOUS. AND	PEAKING CAPACITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY
	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
EXISTING PLANTS														
C. V. PUB. SERVICE	12.2	27	75	80	$ 450	$ 240	$ 690	12.2	27	75	80	$ 450	$ 240	$ 690
BELLOWS FALLS	60	"	218	392	1,308	1,176	2,484	60	"	218	392	1,308	1,176	2,484
VERNON	34.2	"	156	224	936	672	1,608	34.2	"	156	224	936	672	1,608
TURNERS FALLS	64	"	292	420	1,752	1,260	3,012	64	"	292	420	1,752	1,260	3,012
HOLYOKE	(22.8)	0	-	-	-	-	-	22.8	"	104	149	624	447	1,071
TOTALS	170.4		741	1,116	$4,446	$3,348	$7,794	193.2		845	1,265	$5,070	$3,795	$8,865
REDEVELOPED PLANTS														
HOLYOKE								32	"	146	209	876	627	1,503
ENFIELD								28	"	128	183	768	549	1,317
FUTURE PLANTS														
SHARON								57	"	260	372	1,560	1,116	2,676
WEST HARTFORD								35	"	160	229	960	687	1,647
HARTFORD								42	"	192	274	1,152	822	1,974
HART ISLAND								26	"	119	170	714	510	1,224
TOTALS FOR REDEVELOPED AND FUTURE PLANTS								220.0		1,005	1,437	$6,030	$4,311	$10,341
TOTALS								413.2		1,850	2,702	$11,100	$8,106	$19,206

TABLE 41

ANALYSIS OF POWER BENEFITS AVAILABLE FROM WEST CANAAN STORAGE
RESERVOIRS TO DOWNSTREAM PLANTS

RESERVOIR:	66
DRAINAGE AREA:	80 SQUARE MILES
RESERVOIR CAPACITY TOTAL:	47,000 ACRE-FEET - 11 INCHES
RESERVOIR CAPACITY FOR POWER:	26,700 ACRE-FEET - 6.25 INCHES
INCREASED MINIMUM LOW-WATER FLOW:	90 C.F.S.

NAME OF PLANTS	EXISTING PLANTS							COMPREHENSIVE PLAN						
			INCREASED		AVERAGE ANNUAL VALUE OF INCREASED POWER					INCREASED		AVERAGE ANNUAL VALUE OF INCREASED POWER		
	NET HEAD	MIN. FLOW USABLE	PEAKING CAPACITY	ELEC. ENERGY THOUS-AND	PEAKING CAPACITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY	NET HEAD	MIN. FLOW AVAILABLE	PEAKING CAPACITY	ELEC. ENERGY THOUS-AND	PEAKING CAPACITY	ELECTRIC ENERGY	TOTAL CAPACITY & ENERGY
	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS	FT.	CFS	K.W.	KWH	DOLLARS	DOLLARS	DOLLARS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
EXISTING PLANTS														
AM. WOOLEN CO. (EN)	(14.0)	0	-	-	-	-	-	14.0	90	214	306	$1,284	$ 918	$ 2,202
LEBANON & VICINITY	(122.8)	0	-	-	-	-	-	122.8	"	1,890	2,670	11,340	8,010	19,350
" E. & P. CO. NO. 1	17.8	90	272	389	$1,632	$1,167	$2,799	17.8	"	272	389	1,632	1,167	2,799
" " " " " " 2	15.4	"	235	335	1,410	1,005	2,415	15.4	"	235	335	1,410	1,005	2,415
" " " " " " 4	68.0	"	1,040	1,485	6,240	4,455	10,695	68.0	"	1,040	1,485	6,240	4,455	10,695
BELLOWS FALLS (50)	60	"	915	1,310	5,490	3,930	9,420	60.0	"	915	1,310	5,490	3,930	9,420
VERNON	34.2	"	520	746	3,120	2,238	5,358	34.2	"	520	746	3,120	2,238	5,358
TURNERS FALLS	64	"	958	1,395	5,748	4,185	9,933	64	"	958	1,395	5,748	4,185	9,933
HOLYOKE	(22.8)	0	-	-	-	-	-	22.8	"	347	498	2,082	1,494	3,576
TOTALS	259.4		3,940	5,660	$23,640	16,980	$40,620	419.0		6,391	9,134	$38,346	$27,402	$65,748
REDEVELOPED PLANTS														
HOLYOKE								32.	"	487	700	2,922	2,100	5,022
ENFIELD								28	"	426	612	2,556	1,836	4,392
FUTURE PLANTS														
MASCOMA LAKE								101	"	1,535	2,200	9,210	6,600	15,810
BOSTON EXCELSIOR								14.5	"	220	316	1,320	948	2,268
LEBANON E. & P. CO. NO. 2								95	"	1,445	2,070	8,670	6,210	14,880
HART ISLAND								26	"	396	568	2,376	1,704	4,080
TOTALS FOR REDEVELOPED AND FUTURE PLANTS								296.5		4,509	6,466	$27,054	$19,398	$46,452
TOTALS								715.5		10,900	15,600	65,400	46,800	$112,200

TABLE 42

ANALYSIS OF POWER BENEFITS AVAILABLE FROM STOCKER POND STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

Reservoir No.: 63A
Drainage Area: 35 square miles
Reservoir Capacity Total: 20,000 acre-feet - 10.7 inches
Reservoir Capacity for Power: 11,000 " " - 5.88 "
Increased Minimum Low-Water Flow: 36 c.f.s.

Name of Plant	EXISTING PLANTS: Net Head	Min. Flow Usable	INCREASED: Peaking Capacity	Elec. Energy Thous-and	AVERAGE ANNUAL VALUE OF INCREASED POWER: Peaking Capacity	Electric Energy	Total Capacity & Energy	COMPREHENSIVE PLAN: Net Head	Min. Flow Available	INCREASED: Peaking Capacity	Elec. Energy Thous-and	AVERAGE ANNUAL VALUE OF INCREASED POWER: Peaking Capacity	Electric Energy	Total Capacity & Energy
	ft.	cfs	k.w.	kwh	dollars	dollars	dollars	ft.	cfs	k.w.	kwh	dollars	dollars	dollars
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Existing Plants														
Plants in) Claremont)	141	36	1,145	1,270	$6,870	$3,810	$10,680	141	36	1,145	1,270	$6,870	$3,810	$10,680
Bellows Falls	60	36	293	541	1,758	1,623	3,381	60	36	293	541	1,758	1,623	3,381
Vernon	34.2	36	208	308	1,248	924	2,172	34.2	36	208	308	1,248	924	2,172
Turners Falls	64	36	390	578	2,340	1,734	4,074	64	36	390	578	2,340	1,734	4,074
Holyoke	(22.8)	0	0	0	-	-	-	22.8	36	139	206	834	618	1,452
Totals	299.2		2,036	2,697	12,216	8,091	20,307	322.0		2,175	2,903	13,050	8,709	21,759
Redeveloped Plants														
Holyoke								32	36	195	288	1,170	864	2,034
Enfield								28	36	171	252	1,026	756	1,782
Totals for Redeveloped Plants								60		366	540	2,196	1,620	3,816
TOTALS								382.0		2,541	3,443	$15,246	$10,329	$25,575

TABLE 43

ANALYSIS OF POWER BENEFITS AVAILABLE FROM PERKINSVILLE STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

Reservoir: 74
Drainage Area: 142 square miles
Reservoir Capacity Total: 118,000 acre-feet - 15.6 inches
Reservoir Capacity for Power: 84,000 " " - 11.1 "
Increased Minimum Low-Water Flow: 282 c.f.s

Name of Plant	Existing Plants: Net Head	Existing Plants: Increased Min. Flow Usable	Existing Plants: Increased Peaking Capacity	Existing Plants: Increased Elec. Energy Thousand	Existing Plants: Average Annual Value of Increased Power: Peaking Capacity	Existing Plants: Average Annual Value of Increased Power: Electric Energy	Existing Plants: Average Annual Value of Increased Power: Total Capacity & Energy	Comprehensive Plan: Net Head	Comprehensive Plan: Increased Min. Flow Available	Comprehensive Plan: Increased Peaking Capacity	Comprehensive Plan: Increased Elec. Energy Thousand	Comprehensive Plan: Average Annual Value of Increased Power: Peaking Capacity	Comprehensive Plan: Average Annual Value of Increased Power: Electric Energy	Comprehensive Plan: Average Annual Value of Increased Power: TOTAL Capacity & Energy
	ft.	cfs	k.w.	kwh	dollars	dollars	dollars	ft.	cfs	k.w.	kwh	dollars	dollars	dollars
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Existing Plants														
Vt. Hydroelectric.	(22.0)	0	-	-	-	-	-	22.0	282	1,050	1,513	$6,300	$4,539	$10,839
Heald Co.	(9.3)	0	-	-	-	-	-	9.3	"	445	640	2,670	1,920	4,590
Fellow Gear Shaper Co.	(10.1)	0	-	-	-	-	-	10.1	"	482	695	2,892	2,085	4,977
Gilman & Son	(10.5)	0	-	-	-	-	-	10.5	"	502	724	3,012	2,172	5,184
J.T. Slack (3 plants)	(71.3)	0	-	-	-	-	-	71.3	"	3,410	4,920	20,460	14,760	35,220
Lovejoy Tool Co.	(9.5)	0	-	-	-	-	-	9.5	"	454	655	2,724	1,965	4,689
Vermont Snath Co.	(7.7)	0	-	-	-	-	-	7.7	"	367	530	2,202	1,590	3,792
Spgfld. Elec. R.R. Co.	(22.0)	0	-	-	-	-	-	22.0	"	1,050	1,513	6,300	4,539	10,839
Bellows Falls	60.0	282	2,860	4,120	17,160	12,360	29,520	60.0	"	2,860	4,120	17,160	12,360	29,520
Vernon	34.2	"	1,625	2,340	9,750	7,020	16,770	34.2	"	1,625	2,340	9,750	7,020	16,770
Turners Falls	64.0	"	3,045	4,390	18,270	13,170	31,440	64.0	"	3,045	4,390	18,270	13,170	31,440
Holyoke	(22.8)	0	-	-	-	-	-	22.8	"	1,090	1,570	6,540	4,710	11,250
Totals			7,530	10,850	$45,180	$32,550	$77,730	343.4		16,380	23,610	$98,280	$70,830	$169,110
Redeveloped Plants														
Holyoke								32.0	"	1,530	2,210	9,180	6,630	15,810
Enfield								28.0	"	1,340	1,930	8,040	5,790	13,830
Future Plants														
No. Springfield (additional)								9.4	"	450	650	2,700	1,950	4,650
Totals for Redeveloped and Future Plants								69.4		3,320	4,790	19,920	14,370	34,290
TOTALS								412.8		19,700	28,400	118,200	$85,200	$203,400

TABLE 44

ANALYSIS OF POWER BENEFITS AVAILABLE FROM NEWFANE STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

Reservoir:	40A
Drainage Area:	326 square miles
Reservoir Capacity Total:	222,500 acre-feet - 12.8 inches
Reservoir Capacity for Power:	144,200 " " - 8.3 "
Increased Minimum Low-Water Flow:	480 c.f.s.

Name of Plant	EXISTING PLANTS							COMPREHENSIVE PLAN						
	Net Head	INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER			Net Head	INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER		
		Min. Flow Usable	Peaking Capacity	Elec. Energy Thousand	Peaking Capacity	Electric Energy	Total Capacity & Energy		Min. Flow Available	Peaking Capacity	Elec. Energy Thousand	Peaking Capacity	Electric Energy	TOTAL Capacity & Energy
	ft.	cfs	k.w.	kwh	dollars	dollars	dollars	ft.	cfs	k.w.	kwh	dollars	dollars	dollars
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Existing Plants														
W. Dummerston	(20)	0	-	-	-	-	-	20	480	1,625	2,360	$9,750	$7,080	$16,830
Vernon	34.2	480	2,780	4,040	$16,680	$12,120	$28,800	34.2	"	2,780	4,040	16,680	12,120	28,800
Turners Falls	64	"	5,200	7,550	31,200	22,650	53,850	64	"	5,200	7,550	31,200	22,650	53,850
Holyoke	(22.8)	0	-	-	-	-	-	22.8	"	1,860	2,690	11,160	8,070	19,230
Totals	98.2		7,980	11,590	$47,880	$34,770	$82,650	141.0	"	11,465	16,640	$68,790	$49,920	$118,710
Redeveloped Plants														
Holyoke								32	"	2,600	3,780	15,600	11,340	26,940
Enfield								28	"	2,280	3,300	13,680	9,900	23,580
Future Plants														
Williamsville								32	"	2,600	3,780	15,600	11,340	26,940
W. Dummerston (redeveloped)								32	"	2,600	3,780	15,600	11,340	26,940
Brattleboro								58.5	"	4,750	6,900	28,500	20,700	49,200
Totals for Redeveloped and Future Plants								182.5		14,830	21,540	$88,980	$64,620	$153,600
TOTALS								323.5		26,295	38,180	$157,770	$114,540	$272,310

TABLE 45

ANALYSIS OF POWER BENEFITS AVAILABLE FROM PRIEST POND STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

Reservoir No.: 61A
Drainage Area: 19 square miles
Reservoir Capacity Total: 17,200 acre-feet - 17 inches
Reservoir Capacity for Power: 12,700 " " - 12.5 "
Increased Minimum Low-Water Flow: 42.5 c.f.s.

Name of Plant	EXISTING PLANTS							COMPREHENSIVE PLAN						
		INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER				INCREASED			AVERAGE ANNUAL VALUE OF INCREASED POWER		
	Net Head	Min. Flow Usable	Peaking Capacity	Elec. Energy Thousand	Peaking Capacity	Electric Energy	Total Capacity & Energy	Net Head	Min. Flow Available	Peaking Capacity	Elec. Energy Thousand	Peaking Capacity	Electric Energy	Total Capacity & Energy
	ft.	cfs	k.w.	kwh	dollars	dollars	dollars	ft.	cfs	k.w.	kwh	dollars	dollars	dollars
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Existing Plants														
Millers Falls Co.	(28.0)	0	-	-	-	-	-	28.0	42.5	202	292	$1,212	$876	$2,088
Millers Falls Paper Co.	(43.2)	0	-	-	-	-	-	43.2	42.5	312	450	1,872	1,350	3,222
Athol Gas & Electric Co. (Farley)	17.1	42.5	123	178	$738	$534	$1,272	17.1	42.5	123	178	738	534	1,272
Erving Paper Mills Co.	(13.9)	0	-	-	-	-	-	13.9	42.5	100	145	600	435	1,035
Athol Gas & Electric Co. (Wendal)	21.1	42.5	152	220	912	660	1,572	21.1	42.5	152	220	912	660	1,572
Westinghouse Elec. Co.	(7.6)	0	-	-	-	-	-	7.6	42.5	55	79	330	237	567
N. D. Cass	(7.6)	0	-	-	-	-	-	7.6	42.5	55	79	330	237	567
J. W. Moulton	(8.5)	0	-	-	-	-	-	8.5	42.5	61	88	366	264	630
L. S. Starrett	(17.1)	0	-	-	-	-	-	17.1	42.5	123	178	738	534	1,272
Union Twist Drill	(9.0)	0	-	-	-	-	-	9.0	42.5	65	94	390	282	672
Athol Mfg. Co.	(15.0)	0	-	-	-	-	-	15.0	42.5	108	156	648	468	1,116
Mason Parker Co.	(13.9)	0	-	-	-	-	-	13.9	42.5	100	145	600	435	1,035
Turners Falls	64.0	42.5	462	665	2,772	1,995	4,767	64.0	42.5	462	665	2,772	1,995	4,767
Holyoke	(22.8)	0	-	-	-	-	-	22.8	42.5	164	238	984	714	1,698
Totals	102.2		737	1,063	$4,422	$3,189	$7,611	288.8		2,082	3,007	$12,492	$9,021	$21,513
Redeveloped Plants														
Holyoke								32.0	42.5	230	333	1,380	999	2,379
Enfield								28.0	42.5	202	292	1,212	876	2,088
Future Plants														
Upper So. Royalton								70.	42.5	505	730	3,030	2,190	5,220
Lower " "								108.	42.5	777	1,125	4,662	3,375	8,037
Erving								13.5	42.5	97	140	582	420	1,002
Farley								86.	42.5	620	895	3,720	2,685	6,405
Totals for Redeveloped and Future Plants								337.5		2,431	3,515	14,586	10,545	25,131
TOTALS								626.3		4,513	6,522	$27,078	$19,566	$46,644

TABLE 46

ANALYSIS OF POWER BENEFITS AVAILABLE FROM TULLY STORAGE RESERVOIRS TO DOWNSTREAM PLANTS

RESERVOIR NO.: 62A
DRAINAGE AREA: 50 SQUARE MILES
RESERVOIR CAPACITY TOTAL: 48,000 ACRE-FEET - 18 INCHES
RESERVOIR CAPACITY FOR POWER: 33,300 ACRE-FEET - 12.5 INCHES
INCREASED MINIMUM LOW-WATER FLOW: 112 C.F.S.

NAME OF PLANT	EXISTING PLANTS: NET HEAD FT.	EXISTING PLANTS: INCREASED MIN. FLOW USABLE CFS	EXISTING PLANTS: INCREASED PEAKING CAPACITY K.W.	EXISTING PLANTS: INCREASED ELEC. ENERGY THOUS. KWH	EXISTING PLANTS: AVERAGE ANNUAL VALUE OF INCREASED POWER: PEAKING CAPACITY DOLLARS	EXISTING PLANTS: AVERAGE ANNUAL VALUE: ELECTRIC ENERGY DOLLARS	EXISTING PLANTS: AVERAGE ANNUAL VALUE: TOTAL CAPACITY & ENERGY DOLLARS	COMPREHENSIVE PLAN: NET HEAD FT.	COMPREHENSIVE PLAN: INCREASED MIN. FLOW AVAILABLE CFS	COMPREHENSIVE PLAN: INCREASED PEAKING CAPACITY K.W.	COMPREHENSIVE PLAN: INCREASED ELEC. ENERGY THOUS. KWH	COMPREHENSIVE PLAN: AVERAGE ANNUAL VALUE OF INCREASED POWER: PEAKING CAPACITY DOLLARS	COMPREHENSIVE PLAN: AVERAGE ANNUAL VALUE: ELECTRIC ENERGY DOLLARS	COMPREHENSIVE PLAN: AVERAGE ANNUAL VALUE: TOTAL CAPACITY & ENERGY DOLLARS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
EXISTING PLANTS														
MILLERS FALLS CO.	(28.0)	-0	-	-	-	-	-	28.0	112	530	765	$3,180	$2,295	$5,475
MILLERS FALLS PAPER CO.	(43.2)	0	-	-	-	-	-	43.2	112	817	1,180	4,902	3,540	8,442
ATHOL GAS & ELECTRIC CO. (FARLEY)	(17.1)	0	-	-	-	-	-	17.1	112	324	467	1,944	1,401	3,345
ERVING PAPER MILLS CO.	(13.9)	0	-	-	-	-	-	13.9	112	263	380	1,578	1,140	2,718
ATHOL GAS & ELECTRIC CO. (WENDAL)	21.1	112	400	577	$2,400	$1,731	$4,131	21.1	112	400	577	2,400	1,731	4,131
WESTINGHOUSE ELEC. CO.	(7.6)	0	-	-	-	-	-	7.6	112	144	208	864	624	1,488
N. D. CASS	(7.6)	0	-	-	-	-	-	7.6	112	144	208	864	624	1,488
J. W. MOULTON	(8.5)	0	-	-	-	-	-	8.5	112	160	232	960	696	1,656
TURNERS FALLS	64.0	112	1,210	1,745	7,260	5,235	12,495	64.0	112	1,210	1,745	7,260	5,235	12,495
HOLYOKE	(22.8)	0	-	-	-	-	-	22.8	112	430	623	2,580	1,869	4,449
TOTALS	85.1		1,610	2,322	9,660	6,966	16,626	233.8		4,422	6,385	26,532	19,155	45,687
REDEVELOPED PLANTS														
HOLYOKE								32	112	605	875	3,630	2,625	6,255
ENFIELD								28	112	530	765	3,180	2,295	5,475
FUTURE PLANTS														
ERVING								13.5	112	255	368	1,530	1,104	2,634
FARLEY								86.	112	1,620	2,340	9,720	7,020	16,740
TOTALS FOR REDEVELOPED AND FUTURE PLANTS								159.5		3,010	4,348	18,060	13,044	31,104
TOTALS								393.3		7,432	10,733	$44,592	$32,199	$76,791

TABLE 47

SUMMARY OF POWER BENEFITS TO DOWNSTREAM PLANTS FROM CONSERVATION RESERVOIRS AT FLOOD CONTROL DAMS

RESERVOIR		DRAINAGE AREA NET	CAPACITIES OF RESERVOIRS						ANNUAL COST OF POWER STORAGE	EXISTING DEVELOPMENTS								COMPREHENSIVE DEVELOPMENTS						
			FLOOD CONTROL		POWER STORAGE		TOTAL			ACCUMULATED USABLE HEAD	INCREASED USABLE			ANNUAL VALUE OF INCREASED				ACCUMULATED USABLE HEAD	INCREASED USABLE		ANNUAL VALUE OF INCREASED			AVER. COST OF INCREASED ENERGY PER K. W. H.
NO.	NAME		Inches	Acre Feet	Inches**	Acre Feet	Inches	Acre Feet			Minimum Low Water Flow	Peaking Capacity	Yearly Output	Peaking Capacity	Electric Energy	Total Peaking And Energy	Average Cost of Increased Energy per K. W. H.		Peaking Capacity	Yearly Output	Peaking Capacity	Electric Energy	Total Peaking And Energy	
		sq. mi.	in.	a.f.	in.	a.f.	in.	a.f.	dollars	ft.	c.f.s.	kw.	kw.h.	dollars	dollars	dollars	mills	ft.	kw.	kw.h.	dollars	dollars	dollars	mills
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
22A	Victory	66	7.0	24,600	12.5	44,000	17.0	59,800	23,200	263.4	146	5640	8,737	33,840	26,211	60,051	2.7	421.0	10,211	15,031	61,266	45,093	106,359	1.5
27A	Groton Pond	17.3	7.0	6,500	8.75	8,000	14.0	12,900	10,000	244.5	26	1021	1,603	6,126	4,809	10,935	6.2	724.3	3,132	4,743	18,792	14,229	33,021	2.1
28A	South Branch	45	6.0	14,400	6.0	14,400	10.8	25,900	35,000	158.2	48	1283	1,860	7,698	5,580	13,270	18.8	402.9	3,280	4,750	19,680	14,250	33,930	7.4
29A	Gaysville	226	6.5	77,800	5.5	66,300	10.9	131,200	72,600	170.4	220	6350	9,250	38,100	27,750	65,850	7.8	413.2	15,113	22,460	90,678	67,380	158,058	3.2
30A	Ayers Brook	30	6.0	9,800	5.0	8,000	10.0	16,000	18,000	170.4	27	741	1,116	4,446	3,348	7,794	16.1	413.2	1,850	2,702	11,100	8,106	19,206	6.7
66	West Canaan	80	6.0	25,700	6.25	26,700	11.0	47,000	14,600	259.4	90	3940	5,660	23,640	16,980	40,620	2.6	715.5	10,900	15,600	65,400	46,800	112,200	094
53A	Stocker Pond	35	6.0	11,300	5.88	11,000	10.7	20,000	10,000	299.2	36	2036	2,697	12,216	8,091	20,307	3.7	382.0	2,541	3,443	15,246	10,329	25,575	2.9
74	Perkinsville	142	6.0	46,200	11.1	84,000	15.6	118,000	183,000	158.2	282	7530	10,850	45,180	32,550	77,730	16.8	412.8	19,700	28,400	118,200	85,200	203,400	6.4
40A	Newfane	326	6.0	105,000	8.3	144,200	12.8	222,500	128,100	98.2	480	7980	11,590	47,880	34,770	82,650	11.1	323.5	26,295	38,180	157,770	114,540	272,310	3.4
*61A	Priest Pond	19	6.0	6,000	12.5	12,700	17.0	17,200	18,000	102.2	42.5	737	1,063	4,422	3,189	7,611	16.9	626.3	4,513	6,522	27,078	19,566	46,644	2.8
*62A	Tully	50	8.0	21,300	12.5	33,300	18.0	48,000	34,000	85.1	112	1610	2,322	9,660	6,966	16,626	14.6	393.3	7,432	10,733	44,592	32,199	76,791	3.2
47	Knightville	164	4.5	39,300	1.5	13,000	6.0	52,300	46,000	0	44	-	-	-	-	-	-	413.9	3,080	4,400	18,480	13,200	31,680	10.5

*Power benefits and annual costs computed on the basis of a capacity of 12.5 inches for power storage, but it is possible that the industrial demand for increased low water flow in the Millers River may justify a higher capacity.

**Power Storage = (Total Capacity – Flood Control Capacity) + [.25(Total Capacity – Flood Control Capacity)][1]   [1]Max. Value = 3.0" or (Flood Control Capacity - 4.5")

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TABLE 48

ANALYSIS OF THE AMOUNT OF POWER AVAILABLE FROM POSSIBLE NEW POWER DEVELOPMENTS AND THE REDEVELOPMENT OF EXISTING PLANTS AFTER CONSERVATION RESERVOIRS ARE DEVELOPED

Stream	Plant	Served by Reservoir No.	Drainage Area	Head		Additional Reservoir Capacity for Power	Min. Flow		Capacity		Average Annual Power Output	
				Gross	Net		Unreg. **	Total Regul.	Installed	Peaking	Prime Power	Total Power
			Sq.Mi.	Ft.	Ft.	Acre-Feet	c.f.s.	c.f.s.	K.W.	K.W.	Thous.KWH	Thous.KWH
1	2	3	4	5	6	7	8	9	10	11	12	13
Connecticut	Piermont	22A,27A	3,104	37.0	27.5	52,000	1,231	1,403	13,000	6,500	23,000	67,900
*Connecticut	Wilder	22A,27A	3,367	39.5	35.0	52,000	1,280	1,460	17,500	8,700	30,500	95,600
White River	Gaysville	29A,	226	199.0	190.0	66,300	45	220	17,600	7,075	24,800	37,700
White River	Sharon	29A,30A	649	59.0	57.0	74,300	130	385	6,000	4,950	13,000	32,600
White River	West Hartford	29A,30A	683	38.5	35.0	74,300	137	390	4,000	3,100	8,100	21,050
White River	Hartford	29A,30A	708	45.0	42.0	74,300	142	395	5,000	3,750	9,850	26,200
Connecticut	Hart Island	22A,27A,29A, 30A,66	4,573	33.5	26.0	153,000	1,525	2,055	18,500	9,050	31,700	100,500
West River	Newfane	40A	326	135.0	110.0	144,200	65	450	17,900	8,400	29,300	34,100
West River	Williamsville	40A	400		32.0	144,200	80	480	4,200	3,500	9,100	12,150
West River	W. Dummerston	40A	408		52.0	144,200	82	480	6,800	5,650	14,800	20,100
West River	Brattleboro	40A	420		58.5	144,200	84	480	7,600	6,350	16,600	23,300
*Connecticut	Enfield	Above & 53A, 74,61A,62A	9,658	33.7	28.0	438,200	2,562	4,880	35,000	23,100	75,400	214,000
							Totals			90,125	286,150	685,200

* Redevelopments

** Unregulated minimum flows based on 0.2 second feet per square mile of drainage area plus 610 second feet from Connecticut Lakes for Developments located on the Connecticut River.

TABLE 49

GENERAL DATA ON

JUSTIFIED RECREATIONAL DEVELOPMENT

Reservoir	State	Shore Line Miles	Flood Control Pool Area Acres (1)	Conservation Pool Area Acres	Estimated number of Visitors Thru Area Per Summer	Estimated Number of Cottages	Estimated Net Annual Recreational Income	Estimated Annual Recreational Costs (2)
Bethlehem Junction	N. H.	2.7	860	215	225,750	50	117,500(3)	51,500
West Canaan	N. H.	14.7	1,820	1,280	90,000	279	107,000	Cost contained in Power Pool
Stocker Pond	N. H.	18.0	1,150	1,000	100,000	356	134,500	" " " " "
Victory	Vt.	10.0	2,430	2,000	150,000	183	79,000	" " " " "
Groton Pond	Vt.	6.0	655	560	100,000	116	50,500	" " " " "
Union Village	Vt.	3.3	600	280	200,000	47	36,500	" 1,000
Ayers Brook	Vt.	6.2	720	380	25,000	121	45,000	Cost contained in Power Pool
Gaysville	Vt.	25.0	2,330	1,300	40,000	476	170,500	" " " " "
Newfane	Vt.	23.0	3,180	2,270	55,000	437	158,500	" " " " "
Tully	Mass.	11.5	1,750	1,425	100,000	217	86,000	" " " " "
Priest Pond	Mass.	8.4	900	770	100,000	166	68,000	" " " " "
Total, N. H.		35.4	3,830	2,495	415,750	685	359,000	51,500
Total, Vt.		73.5	9,915	6,790	570,000	1,380	540,000	1,000
Total, Mass.		19.9	2,650	2,195	200,000	383	154,000	
GRAND TOTAL		128.8	16,395	11,480	1,185,750	2,448	1,053,000	52,500

(1) Includes Conservation Pool Area.
(2) Generally when conservation capacity is operated for power storage, no additional construction costs for the dam will be necessary to permit recreation use.
(3) Heavy visitor traffic area already developed.

TABLE 49-B

SOURCES OF RECREATION INCOME - NEW HAMPSHIRE

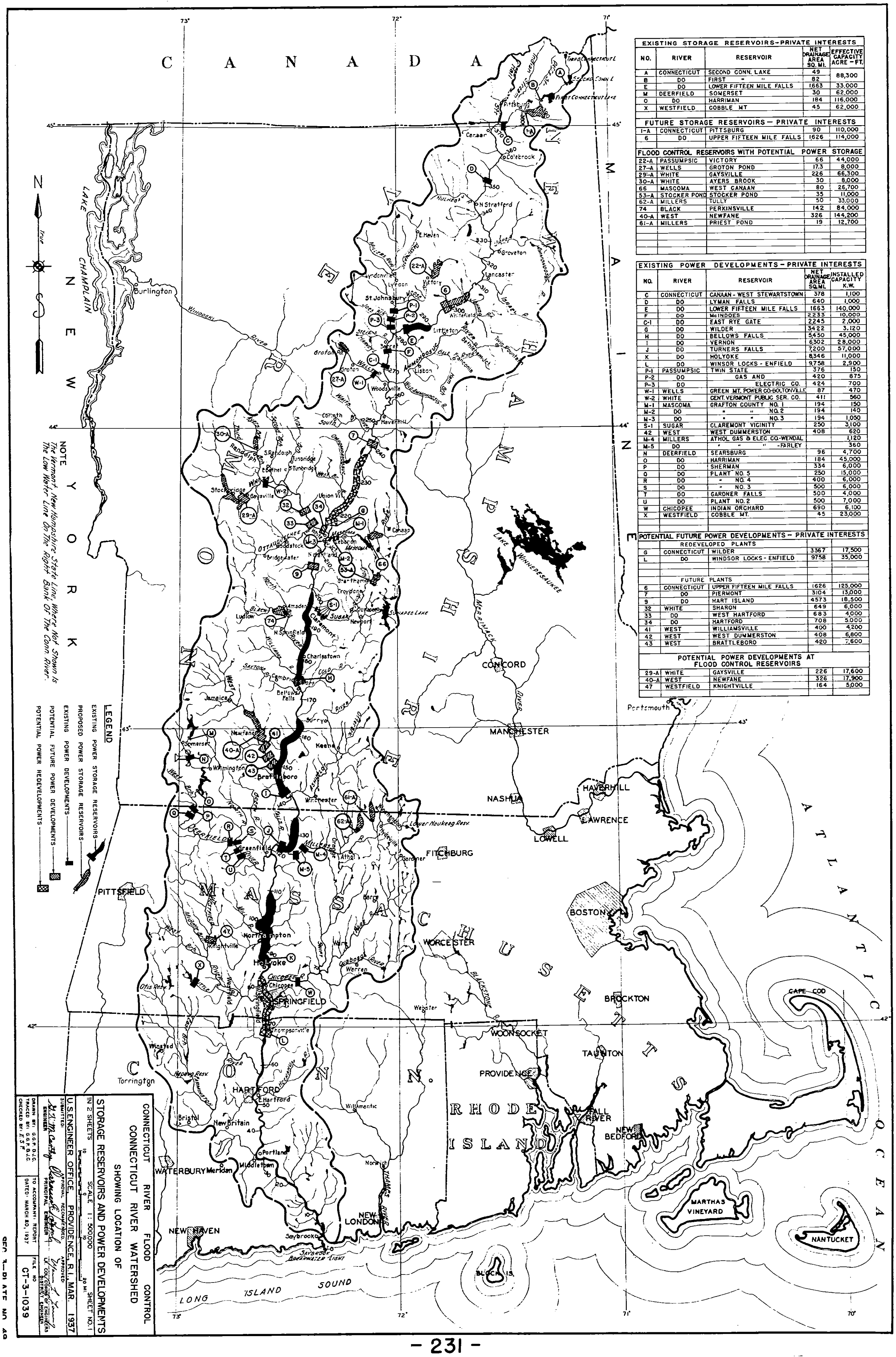
BASED ON NEW HAMPSHIRE PLANNING BOARD TABLE

THE CONNECTICUT RIVER BASIN SECTION OF STATE COMPARED TO TOTALS FOR THE STATE

CLASSIFICATION OF GUEST	NUMBER OF ESTABLISHMENTS		NUMBER OF ACCOMMODATION		PERIOD AVERAGE TURNOVER	NUMBER OF TURNOVERS	TOTAL NUMBER OF GUESTS		AVERAGE LENGTH OF STAY	NUMBER OF VACATION DAYS		AVERAGE EXP. PER DAY	TOTAL EXPENDITURE	
	STATE	C. R. B.	STATE	C. R. B.			STATE	C. R. B.		STATES	C. R. B.		STATE	C. R. B.
SUMMER HOME RESIDENT	12,000	3,000	60,000	15,000	30 DAYS	3	180,000	45,000	30 DAYS	7,400,000	1,850,000	$4.00	$29,600,000	$7,400,000
HOTEL GUEST	270	68	28,000	7,000	7 "	16	448,000	112,000	6 "	2,688,000	672,000	6.00	16,128,000	4,032,000
LODGING HOUSE GUEST	1,200	300	16,000	4,000	14 "	5	80,000	20,000	12 "	960,000	240,000	3.00	2,880,000	720,000
CABIN GUEST	2,800	700	9,000	2,300	3 "	28	252,000	63,000	2 "	504,000	126,000	3.00	1,512,000	378,000
JUVENILE CAMPS	210	52	12,500	3,100	30 "	2	25,000	6,000	30 "	750,000	188,000	5.00	3,750,000	937,000
CAMP GROUNDS	24	6	12,000	3,000	7 "	16	192,000	48,000	6 "	1,152,000	288,000	1.00	1,152,000	288,000
TOTAL REQUIRING ACCOMMODATIONS			137,500	34,400			1,177,000	294,000	11.4"	13,454,000	3,364,000	4.10	55,022,000	13,755,000
TRANSIENTS NOT REQUIRING OVERNIGHT FACILITIES							4,823,000	1,206,000	LESS THAN 1 DAY			1.50	7,234,500 62,256,500	1,809,000 15,564,000
NEW CONSTRUCTION CAPITAL IMPROVEMENT ETC. 8% TOTAL VALUE													8,000,000	2,000,000
MISCELLANEOUS INCLUDING PURCHASE OF EQUIP., LICENSES, ETC.													6,000,000	1,500,000
TOTAL INCOME													$76,256,500	19,064,000

SECTION III

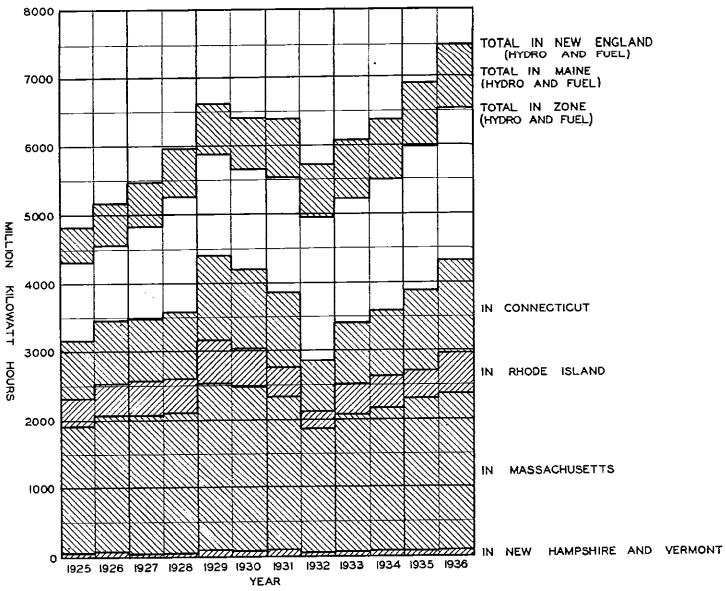
PLATE REFERENCE



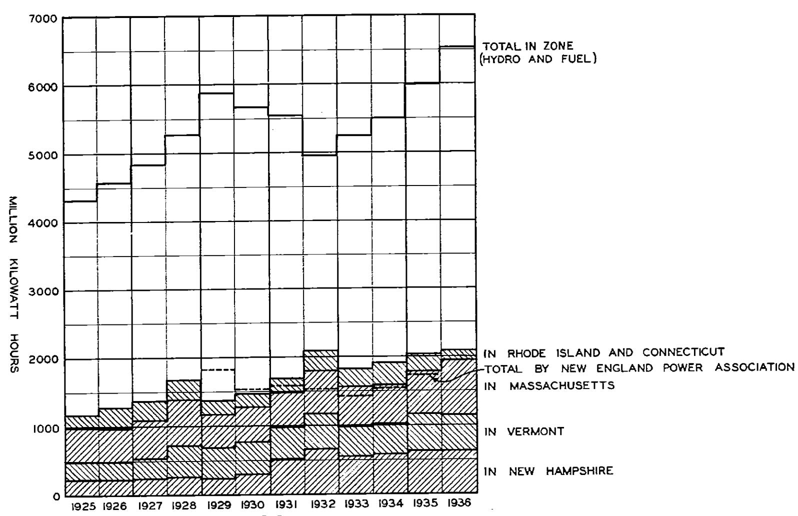
EXISTING STORAGE RESERVOIRS - PRIVATE INTERESTS				
NO.	RIVER	RESERVOIR	NET DRAINAGE AREA SQ. MI.	EFFECTIVE CAPACITY ACRE - FT.
A	CONNECTICUT	SECOND CONN. LAKE	49	88,300
B	DO	FIRST " "	82	
E	DO	LOWER FIFTEEN MILE FALLS	1663	33,000
M	DEERFIELD	SOMERSET	30	62,000
O	DO	HARRIMAN	184	116,000
X	WESTFIELD	COBBLE MT	45	62,000
FUTURE STORAGE RESERVOIRS - PRIVATE INTERESTS				
I-A	CONNECTICUT	PITTSBURG	90	110,000
6	DO	UPPER FIFTEEN MILE FALLS	1626	114,000
FLOOD CONTROL RESERVOIRS WITH POTENTIAL POWER STORAGE				
22-A	PASSUMPSIC	VICTORY	66	44,000
27-A	WELLS	GROTON POND	17.3	8,000
29-A	WHITE	GAYSVILLE	226	66,300
30-A	WHITE	AYERS BROOK	30	8,000
66	MASCOMA	WEST CANAAN	80	26,700
53-A	STOCKER POND	STOCKER POND	35	11,000
62-A	MILLERS	TULLY	50	33,000
74	BLACK	PERKINSVILLE	142	84,000
40-A	WEST	NEWFANE	326	144,200
61-A	MILLERS	PRIEST POND	19	12,700

EXISTING POWER DEVELOPMENTS - PRIVATE INTERESTS				
NO.	RIVER	RESERVOIR	NET DRAINAGE AREA SQ. MI.	INSTALLED CAPACITY K.W.
C	CONNECTICUT	CANAAN - WEST STEWARTSTOWN	378	1,100
D	DO	LYMAN FALLS	640	1,000
E	DO	LOWER FIFTEEN MILE FALLS	1663	140,000
F	DO	McINDOES	2233	10,000
C-1	DO	EAST RYE GATE	2245	2,000
G	DO	WILDER	3422	3,120
H	DO	BELLOWS FALLS	5450	45,000
I	DO	VERNON	6302	28,000
J	DO	TURNERS FALLS	7200	57,000
K	DO	HOLYOKE	8346	11,000
L	DO	WINSOR LOCKS - ENFIELD	9758	2,900
P-1	PASSUMPSIC	TWIN STATE	376	150
P-2	DO	GAS AND	420	875
P-3	DO	ELECTRIC CO.	424	700
W-1	WELLS	GREEN MT. POWER CO-BOLTONVILLE	87	470
W-2	WHITE	CENT. VERMONT PUBLIC SER. CO.	411	560
M-1	MASCOMA	GRAFTON COUNTY NO. 1	194	150
M-2	DO	" " NO. 2	194	140
M-3	DO	" " NO. 3	194	1,050
S-1	SUGAR	CLAREMONT VICINITY	250	3,100
42	WEST	WEST DUMMERSTON	408	620
M-4	MILLERS	ATHOL GAS & ELEC. CO.-WENDAL		1,120
M-5	DO	" " " -FARLEY		360
N	DEERFIELD	SEARSBURG	96	4,700
O	DO	HARRIMAN	184	45,000
P	DO	SHERMAN	334	6,000
Q	DO	PLANT NO. 5	250	15,000
R	DO	" NO. 4	400	6,000
S	DO	" NO. 3	500	6,000
T	DO	GARDNER FALLS	500	4,000
U	DO	PLANT NO. 2	500	7,000
W	CHICOPEE	INDIAN ORCHARD	690	6,100
X	WESTFIELD	COBBLE MT.	45	23,000
POTENTIAL FUTURE POWER DEVELOPMENTS - PRIVATE INTERESTS				
	REDEVELOPED PLANTS			
G	CONNECTICUT	WILDER	3367	17,500
L	DO	WINDSOR LOCKS - ENFIELD	9758	35,000
	FUTURE PLANTS			
6	CONNECTICUT	UPPER FIFTEEN MILE FALLS	1626	125,000
7	DO	PIERMONT	3104	13,000
9	DO	HART ISLAND	4573	18,500
32	WHITE	SHARON	649	6,000
33	DO	WEST HARTFORD	683	4,000
34	DO	HARTFORD	708	5,000
41	WEST	WILLIAMSVILLE	400	4,200
42	WEST	WEST DUMMERSTON	408	6,800
43	WEST	BRATTLEBORO	420	7,600
POTENTIAL POWER DEVELOPMENTS AT FLOOD CONTROL RESERVOIRS				
29-A	WHITE	GAYSVILLE	226	17,600
40-A	WEST	NEWFANE	326	17,900
47	WESTFIELD	KNIGHTVILLE	164	5,000

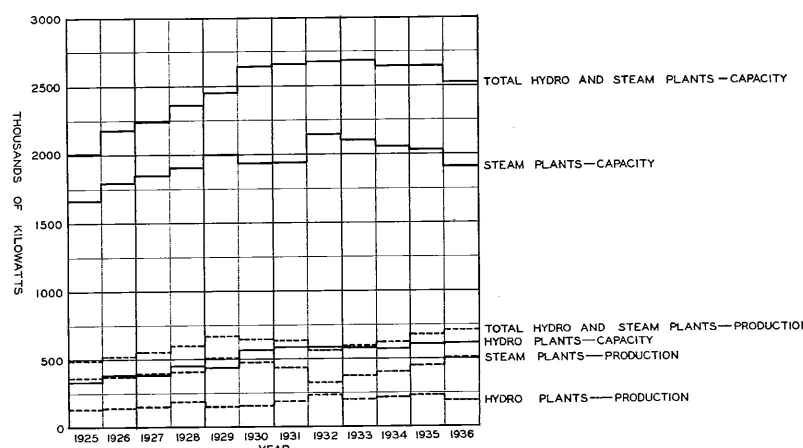
WAR DEPARTMENT



PRODUCTION OF ELECTRIC POWER BY FUELS
IN NEW ENGLAND EXCLUSIVE OF MAINE



PRODUCTION OF HYDRO ELECTRIC POWER
IN NEW ENGLAND EXCLUSIVE OF MAINE



AVERAGE PRODUCTION AND INSTALLED CAPACITIES OF
ELECTRIC POWER PLANTS IN NEW ENGLAND
EXCLUSIVE OF MAINE

CONNECTICUT RIVER FLOOD CONTROL
CAPACITIES AND PRODUCTION
OF
ELECTRIC POWER PLANTS

IN 2 SHEETS | SCALE AS SHOWN | SHEET NO. 2

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAR. 1937

SUBMITTED [signature] ENGINEER | APPROVAL RECOMMENDED [signature] PRINCIPAL ENGINEER | APPROVED [signature] LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DRAWN BY E.S.T. TRACED BY L.W.G. CHECKED BY E.S.F. | TO ACCOMPANY REPORT DATED MARCH 20, 1937 | FILE NO. CT-3-1040

SEC. 3-PLATE NO. 50

CORPS OF ENGINEERS, U.S. ARMY